Performance of Concrete Pavements

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Volume I: Field Investigation

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FOREWORD

This report is part of a four-volume series on "Performance of Concrete Pavements." The goal of this project was to improve design and construction procedures. During the field investigation, phase 303 inservice "experimental" pavement sections, located throughout North America, were surveyed. The sections are termed "experimental," because they were built in individual State studies, to investigate design variables. Fifteen States participated in the study: Arizona, California, Florida, Georgia, Illinois, Michigan, Minnesota, Missouri, New Jersey, New York, North Carolina, Ohio, Pennsylvania, West Virginia, Wisconsin and the Province of Ontario. About one-third of the sections were also surveyed 5 years ago, so some data is available on concrete pavement performance trends. Data is also available on the performance of 96 sections from Europe and 21 sections from Chile. The data collected was entered into the Rigid Pavement Performance (RIPPER) data base, which is available from the Federal Highway Administration.

Volume I summarizes the experimental pavement sections, including the design variables and major performance data. Volume II presents the results of a study of design features. Volume III discusses the newly developed equations for cracking, faulting, spalling, serviceability, and roughness. Volume IV documents the study data and key findings from the European and Chilean studies. Highway engineers will find the reports to be a valuable reference.

Charles J. Nemmers, P.E.
Director, Office of Engineering and
Highway Operations R&D

NOTICE

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16. Abstract

With the goal of improving future concrete pavement design and construction practices, this project evaluated the performance of 303 inservice concrete pavement sections located throughout North America. An extensive field testing program, consisting of pavement condition surveys, drainage surveys, falling weight deflectometer (FWD) testing, coring/boring operations, and roughness testing, was conducted in order to collect the information needed for analysis. Because many of these pavement sections are part of State-level studies on concrete pavements, a range of design variables (e.g., load transfer, slab thickness, joint spacing, drainage) thought to affect concrete pavement performance are present. Over one-third of the sections was evaluated under a preceding FHWA study, meaning that 5-year performance trends are available for some of the sections. Additional pavement performance data are also available for 96 European concrete pavement sections and for 21 Chilean concrete pavement sections. The average age and average cumulative ESAL loadings for the North American sections are 16 years and 7.1 million, respectively, compared to 21 years and 21.8 million for the European sections and 9 years and 5.9 million for the Chilean sections.

This volume describes the field data collection activities that were conducted under the study and also summarizes the individual performance of the 303 concrete pavement sections evaluated. Pertinent design and construction data are given for each project and key performance data are presented. Where available, 5-year performance trends are also summarized.

This volume is the first of four volumes. The other volumes are:

FHWA-RD-95-110 Volume II—Evaluation of Inservice Concrete Pavements
FHWA-RD-95-111 Volume III—Improving Concrete Pavement Performance
FHWA-RD-95-112 Volume IV—Appendixes

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yards	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: Volumes greater than 1000 l shall be shown in m^3.				
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9 or (F-32)/1.8	Celcius temperature	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.71	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

* SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

(Revised September 1993)

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List of Acronyms and Abbreviations

AASHTO	American Association of State Highway and Transportation Officials
AC	Asphalt Concrete
ADT	Average Daily Traffic
AGG	Dense-Graded Aggregate Base
ATB	Asphalt-Treated Base
C_d	Drainage Coefficient
CESAL	Cumulative Equivalent Single Axle Load
COE	Corps of Engineers
COPES	Concrete Pavement Evaluation System
CRCP	Continuously Reinforced Concrete Pavement
CTB	Cement-Treated Base
CSB	Cement-Stabilized Base
C_u	Coefficient of Uniformity
DF	Dry-Freeze
DNF	Dry-Nonfreeze
D_o	Load Plate Sensor
E	Concrete Elastic Modulus
ESAL	Equivalent Single-Axle Load
ESAR	Equivalent Single Axle Radius
FI	Freezing Index
FHWA	Federal Highway Administration
FRC	Fiber Reinforced Concrete
FWD	Falling Weight Deflectometer
GRB	Granular Base
IRI	International Roughness Index
JPCP	Jointed Plain Concrete Pavement
JRCP	Jointed Reinforced Concrete Pavement
k	Modulus of Subgrade Reaction
LCB	Lean Concrete Base
LDF	Load Distribution Factor
LEF	Load Equivalency Factor
LTE	Load Transfer Efficiency
LTPP	Long-Term Pavement Performance
MR	Modulus of Rupture
NOAA	National Oceanic and Atmospheric Administration
PAGG	Permeable Aggregate Base (nontreated)
PATB	Permeable Asphalt-Treated Base
P/C	Pass to Coverage Ratio
PCA	Portland Cement Association
PCC	Portland Cement Concrete
PCTB	Permeable Cement-Treated Base

List of Acronyms and Abbreviations (continued)

PIARC	Permanent International Association of Road Congresses
PSR	Present Servicability Rating
P-value	Probability Value
PVC	Polyvinylchloride
SC	Soil Cement Base
SCS	Soil Conservation Service
SHRP	Strategic Highway Research Program
SEE	Standard Error of the Estimate
SI	International System of Units
TF	Truck Factor
TMI	Thornthwaite Moisture Index
USCS	Unified Soil Classification System
WF	Wet-Freeze
WNF	Wet-Nonfreeze
WWF	Welded-Wire Fabric

1. INTRODUCTION

Background

In 1986, a research study for the Federal Highway Administration (FHWA) was initiated to evaluate the performance and rehabilitation of rigid pavements (*Performance/Rehabilitation of Rigid Pavements [Ripper]*). In terms of rigid pavement performance, the primary objectives of that study were to:

- Evaluate the performance of different rigid pavement design features on in-place pavement sections under similar environmental and traffic loading conditions in each of eight different States.

- Determine the adequacy of available models and design procedures to predict the performance of in-place pavement sections.

- Improve the analysis and design procedures and guidance for the design of rigid pavements to reflect the effects of sealing, drainage, and deflection on pavement performance.

The specific design factors of interest in that study included widened lanes/tied shoulders, thicker slabs, base types, joint design and orientation, load transfer, the presence and amount of reinforcement, shoulder type, subdrainage, and subgrade type.

That study lasted over 4 years and produced a great deal of extremely useful information regarding the performance of concrete pavements, which is fully documented in a six-volume report published by the FHWA.[1-6] Through observations and measurements of the performance of these pavement sections, guidance was developed on the appropriateness of many of the factors noted above, including recommendations on: the effect of base type; the significance of drainage; the relationship between critical joint and slab parameters and slab cracking; the effect of tied shoulders; and the value of load transfer.

One of the most interesting products was the evaluation of a number of analytical tools and models for use in pavement design and evaluation. Using the actual field performance data, evaluations were conducted to assess the accuracy and reasonableness of various pavement analysis programs and pavement performance models, including AASHTO, PEARDARP, PREDICT (COPES), CMS, ILLISLAB, JSLAB, and others. Through the use of the data base developed under that project, as well as data bases from several other research efforts, improved models were developed to predict the performance of different pavement types. A significant study of the cost-effectiveness of many rigid pavement design features was performed using the field data and the prediction models.

While much useful information came from the original study, the findings and results of that study were limited to the pavement designs present in the data base. For example, the predictive equations did not entirely reflect current trends in pavement design practices. Furthermore, the data collected under the original study represent a "snapshot" in the performance life of the pavement section. That is, there was no time series performance data that could provide an indication of the section's rate of deterioration, or how the pavement performed over time. Furthermore, many of the sections that incorporated recent design innovations were too new or had not carried enough traffic for their observed performance to have much meaning.

In order to address many of these deficiencies, the FHWA sponsored a followup study in 1991. Not only were the original 95 sections specified for reinspection, many new sections were added to strengthen and supplement the original data base. A total of 270 concrete pavement sections were manually surveyed and evaluated during 1992. Additional data were available from automated survey methods, yielding a total of 308 concrete pavement sections.

Project Scope and Objectives

This project is a followup to the previously discussed project conducted between 1986 and 1990. One objective of this current study is to reevaluate those 95 pavement sections in order to determine deterioration rates. However, in recognition of the limited number of designs and design sections in the original study, this study is intended to expand the pavement performance data base by incorporating additional concrete pavement sections in order to develop improved guidance on the design and performance of concrete pavements.

The objectives of this project, as stated in the contract, are:

- To reevaluate the 95 projects originally surveyed in 1987 to reveal performance trends and to determine deterioration rates.

- To determine the impact of different pavement types, design features, materials, and construction variables on pavement performance, based on additional data collection and testing, data analysis, and performance evaluations.

- To improve design procedures and performance prediction models for jointed concrete pavements, using an expanded data base. Where possible, evaluate the performance of the various rigid pavement types to provide improved guidance on pavement type selection.

This report describes the results of the efforts undertaken to meet the first objective. It describes the data collection procedures, the sections included in the study, and the proposed data analysis plans. Summaries of the data collected for each of the sections are also provided.

While this project is very similar to its predecessor, a number of "improvements" to the original study have been made possible, if only because of the passage of time. For example, data collected in 1992 is from pavements that are older and have carried more traffic, allowing a time series sequence of data to be available for improved model development.

Progress has also been made in a number of key technological areas that affected how this project could be performed. The original pavements were subjected to an intensive field evaluation that included the hand recording of visible concrete pavement distresses, as well as faulting, crack widths, shoulder drop-offs, and so on. The technology of automated distress gathering equipment has progressed to the point that in some instances automated surveys of pavement surface distresses were performed. The same is true for the roughness survey. The original data were collected with a Mays Meter mounted in a sedan; the roughness data for this project were obtained from profile measurements recorded by a South Dakota-type road profiler.

Sequence of Report

This report consists of four chapters (including this one). Chapter 2 describes the data collection procedures that were followed during the field surveys, including a detailed description of the field testing performed on each section, the office data collection plan, and what procedures were followed to reduce the data into a manipulable form. Chapter 3 provides a description of each of the sections included in the study and presents some of the preliminary performance observations. Chapter 4 provides a brief summary of this report. A complete summary of all data elements for each section can be found in appendix A of Volume IV.

2. DATA COLLECTION PROCEDURES

Introduction

Data collection is the most critical aspect of this project. A wide range of data elements were collected for this study to allow a complete and thorough analysis of the effects of such factors as pavement design, traffic, and climatic conditions on the performance of concrete pavements. In addition to being used in these analyses, the data collected will be made available for future studies in the same manner as the Strategic Highway Research Program (SHRP) Long Term Pavement Performance (LTPP) data. Hence, the collection of accurate pavement performance data, as well as project information, is of critical importance to the success of this project.

Project information was assembled before the start of any field work, primarily through the review of project reports. This information was used in the selection of projects for inclusion in this study and in the planning of the field work. The field work consisted of distress surveys, drainage surveys, surface profile, destructive testing (coring and boring), and falling weight deflectometer (FWD) testing.

Several steps were taken to ensure the reliability of the data collected and consistency in the data collection process. The SHRP LTPP data collection guide was followed to ensure that all key data elements were collected.[7] In the field, the pavement distress data were identified and quantified according to reference 8. This distress identification manual provides a means of collecting consistent distress data from survey to survey, and its use also ensures that the collected data are consistent with data collected for the LTPP studies.

The composition of the crews participating in the field also helped to ensure consistency. The field survey was completed by two survey crews, each consisting of two engineers, one of which was a senior project engineer. The senior project engineers heading the survey crews remained the same throughout the project, and between them were involved on all of the field surveys. This ensured the consistency in data collection process. In addition, each survey crew was supplied with an identical set of equipment to minimize equipment variability as a potential source of inconsistency.

A comprehensive data collection plan was devised for this project to ensure that all necessary data were collected in the most efficient and effective manner. In most cases, the data collection activities represented a "one-shot" opportunity to collect all the needed data, so careful planning was absolutely essential. Furthermore, since the data collection effort involved several subcontractors, as well as the participating State Highway Agencies, the coordination of the field data collection activities was particularly important. The data collection plan clearly outlined the research team's approach to all aspects of the data collection process and indicated the roles and assignments of all participating organizations.

Data Elements

The data collected under this project can be classified under the following categories: inventory data, monitoring (distress) data, materials testing data, traffic data, climatic data, maintenance data, and rehabilitation data. These categories are essentially the same groupings that are employed by the SHRP LTPP project. Each group is made up of a number of data elements that are needed to conduct the studies proposed under this project. In a few instances, the actual data could not be determined from the field tests. For those data items, either suitable replacements or estimates will be determined by the research team; all such replacements will be flagged in the data base to indicate that they are estimates.

Office Data Collection

The office data collection activities consisted of gathering that information needed prior to the actual field testing, and those items that could not be obtained out in the field. Much of this work had to be completed before any field work could begin. Among other activities, the office data collection involved collecting:

- Project location information.
- Design and construction information.
- Subgrade soil information (obtained from county soil maps).
- Maintenance and rehabilitation records.
- Past performance data, if available (condition surveys, FWD data, and so on).
- Historical traffic records.
- Climatic information, including both historical (published by the National Weather Service) and specific climatic information at the time of construction.

This work consisted primarily of reviewing project reports and data obtained from other sources. Much of the information on the original 95 sections, including project information and the previous field survey data, were obtained from the project files and reports from the previous study (reference 4). The office data collection continued following the field data collection to identify any critical missing data elements and to ensure that all data needed in the study is compiled.

Field Data Collection

The field data collection activities represent a major portion of the data collection efforts in this project. This work consisted of conducting both manual and automated surveys of the pavement sections, as well as destructive and nondestructive field testing. The field testing program consisted of conducting FWD testing, coring, and South Dakota (SD) Profiler testing. Table 1 lists the projects included in this study by environmental zone. In all, 270 sections at 50 projects were evaluated using manual survey methods. The general location of these projects is shown in figure 1.

Table 1. List of sections included in the study.

Env. Zone	Project	Location
Dry-Freeze	MN 1	EB/WB I-94, Rothsay, MN
	MN 2	EB I-90 Albert Lea, MN
	MN 3	EB I-90 Austin, MN (new)
	MN 4	TH 15 New Ulm, MN (new)
	MN 5	EB/WB I-94, Rothsay, MN (Control)
	MN 6	TH 15 Truman, MN (new)
	MN 7	EB/WB T.H. 36, Roseville, MN
Dry-Nonfreeze	AZ 1	RT 360 Phoenix, AZ
	AZ 2	EB I-10 Phoenix, AZ (new)
	CA 1	NB/SB I-5 Tracy, CA
	CA 2	EB I-210 Los Angeles, CA
	CA 6	SB RT 14, Solemint, CA
	CA 7	NB I-5 Sacramento, CA (Control)
	CA 8	NB US 101 Thousand Oaks, CA
	CA 9	NB/SB I-680, Milpitas, CA
	CA 11	SB I-5, Sacramento, CA
Wet-Freeze	IL 1	EB/WB US 50, Carlyle, IL
	IL 2	EB/WB US 20, Freeport, IL
	MI 1	EB/WB US 10, Clare, MI
	MI 3	WB I-94 Marshall, MI
	MI 4	NB I-69 Charlotte, MI
	MI 5	EB I-94 Paw Paw, MI
	MI 6	WB Davison Freeway, Detroit, MI
	MO 1	NB I-35, Bethany, MO
	NJ 2	NB RT 130 Yardville, NJ
	NJ 3	SB RT 676 Camden, NJ
	NY 1	RT 23 Catskill, NY
	NY 2	WB I-88 Otego, NY
	OH 1	SB RT 23, Chillicothe, OH
	OH 2	EB/WB SR2, Vermilion, OH
	ONT 1	HWY 3N Ruthven, ONT
	ONT 2	SB HWY 427, Toronto, ONT
	PA 1	NB/SB TR 66 and EB RT 422 (PA 1-1) Kittanning, PA
	WI 1	WB I-90, Stoughton, WI
	WI 2	EB USH 18/151, Mt. Horeb, WI
	WI 3	EB & WB STH 14, Middleton, WI
	WI 4	SB STH 164, Waukesha, WI
	WI 5	EB STH 50, Kenosha, WI
	WI 6	EB STH 29, Green Bay, WI
	WI 7	EB USH 18/151, Barneveld, WI
	WV 1	NB/SB I-77, Charleston, WV
Wet-Nonfreeze	CA 3	NB/SB US 101, Geyserville, CA
	CA 10	US 101, Ukiah, CA
	FL 2	NB I-75 Tampa, FL
	FL 3	SB I-75 Tampa, FL
	FL 4	NB US 41, Ft. Meyers, FL
	GA 1	SB I-85, Newnan, GA
	GA 2	SB I-85, La Grange, GA
	NC 1	NB/SB I-95 Rocky Mount, NC
	NC 2	NB I-85 Greensboro, NC

Figure 1. General location of sections included in the study.

Traffic control for the field testing activities was provided by the participating State Highway Agencies. The excellent cooperation of the participating State Highway Agencies and the Ontario Ministry of Transportation was invaluable to the successful completion of the field data collection. Although no weather-related delays were encountered—with one notable exception at ONT 1, where the core and FWD testing took four attempts to complete due to the weather—the final arrangements for the traffic control had to be made on short notice on many occasions. The participating highway agencies graciously accommodated the needs of the survey team, allowing the field surveys to be completed without any problems.

Manual Condition Survey

The manual condition survey at each site consisted of four types of surveys: a distress survey, a drainage survey, a photographic survey, and Present Serviceability Rating (PSR). On distress surveys, all visible distresses were either mapped or noted, and various measurements were taken. The objective of conducting the drainage survey was to assess the overall drainage characteristics of each section. In addition to sketching and recording all of the data collected, a photographic survey was conducted to document all significant features of each section, including the typical slab condition, typical distresses, unusual distresses, shoulder condition, and the condition of the drainage ditches. The field data collection forms used to record the data are shown in figures 2 and 3.

The typical procedure followed in the manual condition survey was to first mark off the sections to be surveyed, and then conduct the distress survey, followed by the drainage and photographic survey. The PSR was performed at the end of the day, after all other surveys had been completed. The distress survey was conducted by both members of the survey crew as a team—one person taking measurements, and the other mapping and recording the observed distresses in the field notebook. Following the distress survey, one member of the crew conducted the drainage survey while the other performed the photographic survey.

Distress Survey

Location Information

Every effort was made to record the exact location of each pavement section so that they could be found easily in the future. This is an extremely important task that allows continued monitoring of the pavement sections. That same effort made during the previous study allowed the original 95 sections to be easily located.

The location information was recorded at two levels: project and section. The project-level information was intended to be used for locating the general location of the pavement sections at highway speeds. This is extremely valuable information that allows the test sections to be located without wasting time. Either starting milepost or, if mileposts were not available, the direction and distance from the nearest fixed object (such as an overpass) were used for this purpose.

Field Survey: ***General Information***

Project ID: ________________

ERES ID: ________________

Date of Survey (mm/dd/yy): ___/___/___

Surveyors' Initials: _____/_____/_____

Test Section Location:

	Milemark (MP)	Station (STN)	Section Length, ft
Start Point		+	
End Point		+	

If no MP or STN: Distance from the nearest landmark: __________ ft

Direction FROM the landmark: Line of Traffic / Against Traffic

Landmark description (type/name of structure/interchange/crossroad):

____

____

Shoulder: **General Condition:** Good / Fair / Poor / Failed

Shoulder Surface Type

1. Turf
2. Granular
3. AC
4. Concrete
5. Surface Treatment
6. Other:__________

	Type	Width
Outside Shoulder:		ft
Inside Shoulder:		ft

Contraction Joint: Joint Spacing: _____._ ft Skewed? y / n

Random Spacing: ____-____-____-____ ft ft/Lane: ______

Sealant Type: None HP SI Preform Other: __________

Longitudinal Joint: Method Used to Form Centerline Joint: Saw Cut / Plastic Insert

Sealant Type: None HP SI Preform Other: __________

Roughness and Serviceability:

Roughness Measurement Device Used:

____

		Lane Number* 1	Lane Number* 2	Lane Number* 3
Roughness Index	Trial 1			
	Trial 2			
Roughness Measurement Speed (mph)				
Present Serviceability Rating (mean)				

*Lane 1 is outer lane, Lane 2 is next lane 1, etc.

Figure 2. Field survey form — general information collection form.

To Be Sketched	Note on Sketch	
All Cracking	LF	L M H
Longitudinal Joint Spalling		L M H
Longitudinal Faulting	in	
Crack Faulting	in	
Scalling/Map Cracking	Area	L M H
Patches/Replaced Slabs	Dimensions	
Improper Joint Construction Cracks	LF	
Missawed Joints (Bonded OL)	LF	
Blowups	BU	
D-Cracking	D	L M H
Reactive Aggregate	RA	
All shoulder distresses		

Field Survey: *Data Collection Form*

Date of Survey (mm/dd/yy): ___/___/___

Surveyors' Initials: DGP / HTY / ______

Project ID: ____________

ERES ID: ____________

Page No: ______ of ______

Transverse Joint Type Code
1. Contraction Joint
2. Construction Joint
3. Patch Approach Joint
4. Pressure Relief Joint

	OUTER LANE						INNER LANE					
STATION												
Trans Joint Type (Code)												
Transverse Joint Spalling	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H
Transverse Joint Seal Damage	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H
Longitudinal Joint Seal Damage	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H
D-Cracking	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H
Pumping	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H
Patch/Slab Replacement Deterioration	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H
Slab Deterioration Adjacent to Patch	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H
Lane to Shoulder Dropoff (in)												
Lane to Shoulder Separation (in)												
Faulting (in)												
Joint Width — Outer Lane Only (in)												
Joint Depth (in)												

Figure 3. Field survey form—pavement data collection form.

The section-level information was intended to be used to establish the exact location of the test sections. Whenever possible, the station numbers stamped in the pavement were used for this purpose. If permanently stamped station numbers were not available, the direction and distance from a fixed object (typically bridge overpass) was used. During this survey, the mapped distresses, core locations, and other distinguishing marks on the pavement (such as full-depth repairs) that were documented during the previous survey provided further help in locating the original 95 sections.

As in the previous study, the survey sections of the new projects were located such that the sections are representative of the project and safe to survey. The factors considered in locating the survey sections include:

- Horizontal curves less than 3° and vertical grades less than 4 percent.
- A minimum of cut/fill transitions, either longitudinally or horizontally.
- No culverts, pipes, or other substructures within the section.
- Uniform traffic flow throughout the project.
- Any other factors that may in any way compromise the safety of the field survey teams.

Whenever possible, a minimum of a 1,000-ft (305-m) long section was selected; shorter sections had to be selected in some cases because of the factors listed above.

The location of features of interest, such as distress and other information, were tracked using relative stationing. The beginning of each section was assigned the station number 0+00 and all referencing within that section was done using the station number that increased in the direction of traffic. At the beginning of the survey, the starting location of the section was painted on the pavement and the length to survey was marked off with a measuring wheel. The measured section length was then recorded and the end of the section was painted on the pavement. On jointed concrete pavement sections, the survey sections were always selected to start and end at a joint; hence the actual section length varied slightly from the length planned to be surveyed. The measured section length was later used in the calculation of the roughness index and in normalizing the distress quantities to a per mile basis.

Measurements

The measurements taken as a part of the manual distress survey included:

- <u>Faulting</u>. Joint faulting was measured at every transverse joint in the outer lane and at medium- and high-severity cracks. The faultmeter used in this survey has a dial gauge that allows the faulting to be measured to the nearest 0.01 in (0.25 mm). This device consists of an 18-in (457-mm) long, aluminum base plate with two footings—one at one end of the device and the other placed in the middle, about 11 in (280 mm) from the first footing—and a cylindrical fixture at the overhanging end (the end without the footing) that allows a slotted rod to move freely in the vertical direction. The dial gauge is attached at the top of the slotted rod in such a way that the gauge registers the height difference from the zero-position. The cylindrical fixture has a screw lock that can be used to lock the moveable cylinder in any position. The device is calibrated by setting the faultmeter on top of a 4-ft (1.2-m) level, releasing the screw-lock, and then setting the dial gauge to zero. The probe-end of the slotted cylinder and the middle footing are spaced 6 in (152 mm) apart. A measurement is taken by setting the middle footing and the probe-end equidistant from the joint (or crack), releasing the screw lock, letting the probe rest on the pavement surface, locking it in place, and then reading the dial gauge. The faultmeter was calibrated before each survey, and all faulting measurements were taken 12 in (305 mm) from the edge of the pavement.

- Joint width. Transverse joint widths were measured with a 6-in (152-mm) stainless steel rule to the nearest 0.05 in (1.3 mm). A minimum of two measurements were taken (once at the beginning of the section, and once at the end). Additional measurements were taken if any anomalies were observed.

- Joint depth. Transverse joint depths were measured whenever the joint width was measured. This was accomplished by forcing a screwdriver in the joint until the bottom of the joint reservoir is reached and then measuring the depth of penetration with a 6-in (152-mm) stainless steel rule. The measurement was taken to the nearest 0.05 in (1.3 mm).

- Shoulder dropoff. Lane-shoulder dropoff was measured with the faultmeter to the nearest 0.05 in (1.3 mm). A minimum of two measurements were taken (once at the beginning of the section, and once at the end).

- Shoulder separation. Lane-shoulder separation was measured whenever the shoulder dropoff was measured with a 6-in (152-mm) stainless steel rule to the nearest 0.05 in (1.3 mm). If the shoulder joint was not sealed, then the dirt and other debris were first cleared, using a screwdriver, until the vertical face of the shoulder was exposed and the separation of the vertical face from the pavement edge was measured.

- Slopes. Transverse slopes of all lanes and the shoulders, and the longitudinal slopes of each pavement section were measured with either a 4-ft (1.2-m) or 2-ft (0.6-m) level that has a bubble slope indicator. Both types of levels indicate the slope on an inches (mm) of vertical height difference per ft (m) of horizontal distance basis. The slopes were measured to the nearest 1/16 in (1.6 mm) of this unit at three locations—usually at the beginning, middle, and end of section—and recorded on the drainage survey sheet (figure 4).

Field Survey: *Drainage Information*

Project ID: ____________
ERES ID: ____________

Date of Survey (mm/dd/yy): ___/___/___
Surveyors' Initials: ___/ HTY /___

Slope Measurements:

	Station	Slope	
		Outer	Inner
Longitudinal Slope (nearest 1/16") 3 measurements, equally spaced along project.	+	/	/
	+	/	/
	+	/	/
Transverse Slope (nearest 1/16") 3 measurements, equally spaced along project.	+	/	/
	+	/	/
	+	/	/
Shoulder Slope (nearest 1/16") 3 measurements, equally spaced along project.	+	/	/
	+	/	/
	+	/	/

Cut/Fill and Ditch Line Depth:

Circle, if Cut/Fill Depth Uniform	Cut/Fill Depth	Station(s)		Depth of Ditch Line
1.	Fill > 40 ft	+	+	ft
2.	Fill 16 - 40 ft	+	+	ft
3.	Fill 6 - 16 ft	+	+	ft
4.	At Grade (5 ft fill to 5 ft cut)	+	+	ft
5.	Cut 6 - 15 ft	+	+	ft
6.	Cut 16 - 40 ft	+	+	ft
7.	Cut > 40 ft	+	+	ft

Lane/Shoulder Joint Integrity:

	Outer Shoulder	Inner Shoulder
Sealant Damage	N L M H	N L M H
Blowholes	N L M H	N L M H
Sealant Type	None HP SI Preform Other:	

Subsurface Drainage (visual):

Type of drainage system present: ____

1. None.
2. Longitudinal Drains.
3. Transverse Drains.
7. Other: ____________

Indicators of Poor Drainage:

Cattails or willows growing in ditch: y / n
Drainage outlets clogged: y / n
Drainage outlets below ditch line: y / n
Non-continuous cross section, crown to drainage ditch: y / n
Pumping: N L M H
Other: ____________________

Figure 4. Drainage survey sheet.

Mapped Distresses

An important part of the manual distress survey was to map all visible distresses. Before each field trip, field survey notebooks were prepared that contained sufficient field data sheets for each section to be surveyed. As a part of the preparation work, the relative station numbers were marked on the survey sheets and the transverse joints were drawn in to scale.

The following distresses were drawn on the sheets, approximately to scale, with an accompanying note to indicate the distress severity:

- Cracks (transverse, longitudinal, and compression cracks).
- Corner breaks.
- Spalling.

In addition, all patched areas were drawn to scale on the survey sheets and the patch material was identified. Areas of grinding and grooving, as well as all core holes (both existing and those taken during this study) and any other permanent, distinguishing marks on the pavement were either noted or drawn to scale on the survey sheets. All station numbers stamped on the pavement were correlated to the relative station numbers used within the section and recorded. Fixed objects located within the section boundaries—such as signposts, light posts, bridges, ramps, and drainage structures—were also noted to provide further reference of the section location.

The CRCP sections surveyed were not mapped. Instead, within a designated 1000-ft (305-m) section, the total number of transverse cracks were counted along with the number of deteriorated cracks, punchouts, and other failed areas recorded. In addition, crack widths were measured on several transverse cracks. Finally, a windshield survey of a 2 to 3-mi (3.2 to 4.8 km) segment of the CRCP projects was conducted during which pavement failures (deteriorated cracks, punchouts, patches of punchouts) were counted.

Evaluated Conditions

Several conditions at transverse and longitudinal joints were evaluated and recorded on the survey sheets. These include the type and condition of the joint seal and the extent of transverse joint spalling. The spalling at the transverse joints was evaluated and the level of deterioration was rated as NONE, LOW, MODERATE, or HIGH. Pumping at the transverse joint, as well as the joint seal damage at both transverse (including transverse patch joints) and longitudinal joints were also evaluated and rated similarly. The highest level of pumping that occurred within a section was assigned as the overall pumping for the section. If the joints were not sealed, they were rated as HIGH severity deterioration.

The same evaluation scheme was used to rate the condition of all full-width patches. The condition of both the patches and the concrete adjacent to the patches was rated and noted in the appropriate box on the survey sheet (see figure 3).

Noted Conditions

The surface condition of the pavement was typically noted on a project-wide basis. Any material problems, such as D-cracking and reactive aggregate, were noted for the entire section if all areas were similarly affected. Localized surface distresses, such as popouts and areas of crazing and scaling, were also noted.

Drainage Survey

The drainage survey was performed to gather information on the drainage characteristics of the pavement section. The form shown in figure 4 was used in collecting this data. In addition to the slope measurements described earlier (*Measurements*), the following entries were made on the drainage survey form:

- Cut/Fill and Ditch Line Depth. The cut or fill depth and the depth to the ditch line were estimated and recorded in this section. If these depths are uniform throughout the section, one value for each was entered for the section. If, however, the variation in the landscape was significant from one part of the section to another, the representative values for each segment were recorded along with the beginning and ending stations of the segments.

- Lane/Shoulder Joint Integrity. The sealant type was recorded and the condition of the joint sealant at both the outer and inner shoulder joints were rated and recorded as NONE, LOW, MODERATE, or HIGH. A rating of NONE meant that the sealant was in excellent condition; a rating of HIGH meant that the sealant was either in poor condition or nonexistent. The condition of the joint sealant between the mainline pavement and the shoulder is an important indicator of the drainage condition of the section. Asphalt concrete (AC) shoulders were examined for the presence of blowholes at the lane-shoulder interface. These are semicircular areas that may be present at the shoulder joint where the AC has been removed by the force of a powerful expulsion of free water (along with some of the subgrade and subbase material) from underneath the pavement slab.

- Subsurface Drainage. The presence of any subsurface drainage appurtenances (e.g., outlet pipes) were noted on the survey sheets, along with their condition.

- Indicators of Poor Drainage. The survey form provides a checklist of items that may indicate a poor drainage condition. These items include the presence of cattails or willows growing in the ditches, clogged drainage outlets, drainage outlets located below the ditchline, and signs of pumping. Where present, the transverse drain outlets were examined to determine whether they were able to function effectively.

Poor drainage can be indicated by cattails or willows growing nearby, as these plants are commonly found where there is an adequate water supply. However, the primary indicator of poor subsurface drainage is pumping. The most common sign of pumping is the staining of the shoulder by the fine sediment ejected between the pavement and the shoulder. This phenomenon is usually observed at the transverse joints. As previously noted, pumping severity was rated at the section-level. In addition, signs of pumping were noted on the distress survey form at each transverse joint.

Photographic Survey

The photographic survey was performed as a part of the manual condition survey to further document the condition of the pavement at the time of survey. This consisted of taking 35-mm, color slides of the sections and recording the description of each slide on the form shown in figure 5. The number of photos taken varied from section to section, but the general approach to the survey was the same for all sections; the photo record of a pavement section consists of the following slides:

- Section ID. The section number painted on the pavement was photographed for identification purposes.
- Overview. The first set of slides was shot from the beginning of the section facing toward the end of the section. They show a progression from the outer drainage ditch to the outer shoulder, the mainline pavement, the inner shoulder, and the inner drainage ditch.
- Typical features. The next set of slides illustrate the typical slab, typical transverse joints, typical lane-shoulder and longitudinal joints, and visible drainage features.
- Distresses. This set of slides document the typical distresses and the most notable distresses of the section.
- Overview facing back. The last slide in each section was taken from the end of the project along the centerline of the pavement, facing back toward the start of the section.

The photo record consists of 12 to 18 slides for the sections in fairly good condition. More slides were taken, as many as 36, if the section was badly deteriorated in order to fully document the pavement condition.

Present Serviceability Rating (PSR)

A PSR was performed on all lanes being surveyed. One pass was made over each lane at highway speeds and members of the survey team rated the pavement. The average of the ratings given by each member was then reported. The PSR values from the field survey are summarized in appendix A of Volume IV.

SONORPPR PHOTO LOG

Section ID:________
Photographer:_______

Photo #	Sta	Description	Evaluation
1	+		
2	+		
3	+		
4	+		
5	+		
6	+		
7	+		
8	+		
9	+		
10	+		
11	+		
12	+		
13	+		
14	+		
15	+		
16	+		
17	+		
18	+		
19	+		
20	+		
21	+		
22	+		
23	+		
24	+		
25	+		
26	+		
27	+		
28	+		
29	+		
30	+		
31	+		
32	+		
33	+		
34	+		
35	+		
36	+		
37	+		

Figure 5. Photographic record form.

Automated Distress Survey

An automated distress survey was performed on many of the sections using the PASCO automated survey device. In general, all projects consisting of multiple sections were surveyed using this method. The surveys are conducted by photographing the pavement using 35-mm film and then later analyzing the film to provide the distress data. The filming is performed at night to provide a controlled lighting condition, and at vehicle speeds of 6 to 38 mi/h (10 to 60 km/h). The survey equipment monitors the vehicle speed and automatically adjusts the rate of filming and illumination to provide a constant rate of coverage of filming and constant lighting condition. One pass is made on each lane being surveyed, and each pass of the vehicle films a strip of pavement 16.7-ft (5-m) wide.

The film from the field survey is later analyzed in the laboratory to extract the distress data, which is accomplished manually by reviewing the film on a computer-aided work station. The operator identifies the distressed area, along with the type and severity of distress, and the computer records the information. Through this process, the severity and quantity of the following types of distresses are obtained:

- Corner breaks.
- D-cracking.
- Longitudinal cracking.
- Transverse cracking.
- Transverse joint seal damage.
- Transverse joint spalling
- Longitudinal joint spalling
- Scaling.
- Popouts.
- Blowups.
- Lane-to-shoulder separation.
- AC patch/patch deterioration.
- PCC patch/patch deterioration.
- Water bleeding and pumping.

In addition to the distress summaries, the data analysis tools can generate a distress map that summarizes this information graphically.

The projects surveyed by automated methods are summarized in table 2. The automated survey was conducted continuously over the entire experimental section in each project, rather than just over the selected sample unit sections.

Field Testing

Several types of field testing were conducted on selected test sections to determine the in-place material properties and serviceability. These include:

- Deflection testing, conducted with the FWD, to determine the concrete modulus of elasticity (E), dynamic modulus of subgrade reaction on top of the base (k), and load transfer efficiency (LTE) at both transverse joints and shoulder joints (if a tied PCC shoulder is present).

- Coring and boring to determine layer thicknesses, joint condition, base and subgrade properties, and splitting tensile strength of concrete cores.

Table 2. Listing of projects surveyed by automated methods.

Project	Location	No. of Sections Surveyed	Date Surveyed
AZ 1	SR 360, Phoenix, AZ	6	Sep 91
CA 1	I-5, Tracy, CA	16	Sep 91
CA 3	US 101, Geyserville, CA	10	Sep 91
CA 9	I-680, Milpitas, CA	6	Sep 91
FL 4	US 41, Ft. Meyers, FL	11	Jul 92
MI 1	US 10, Clare, MI	15	Dec 92
MN 1	I-94, Rothsay, MN	24	Mar 92
MN 2	I-90, Albert Lea, MN	4	Mar 92
MN 7	TH 36, Roseville, MN	24	Mar 92
MO 1	I-35, Bethany, MO	8	Apr 92
NJ 3	I-676, Camden, NJ	2	Apr 92
NY 1	RT 23, Catskill, NY	6	Jul 92
NY 2	I-88, Otego, NY	4	Jul 92
NC 1	I-95, Rock Mount, NC	9	Jul 92
OH 1	US 23, Chillicothe, OH	8	Oct 92
OH 2	SR 2, Vermilion, OH	39	Oct 92
ONT 1	HWY 3N, Ruthven, ONT	4	Oct 91
PA 1	RT 66 & RT 422, Kittanning,	9	Jul 92
WI 1	I-90, Stoughton, WI	3	Mar 92
WI 2	US 18/151, Mt. Horeb, WI	5	Mar 92
WI 3	US 14, Middleton, WI	3	Mar 92
WI 4	STH 164, Waukesha, WI	6	Mar 92
WI 5	STH 50, Kenosha, WI	6	Mar 92
WI 6	US 29, Green Bay, WI	4	Mar 92
WI 7	US 18/151, Barneveld, WI	10	Mar 92

- South Dakota (SD) Profiler testing to measure pavement roughness (recorded in International Roughness Index [IRI] units).

In all cases, the field testing was scheduled to coincide with the manual condition survey. This scheduling ensured that a senior project engineer was present at all field testing trips to address any problems that might develop. In addition, this arrangement gave the project engineer the flexibility to increase or decrease the amount of testing as dictated by the field conditions. On a number of occasions, adjustments had to be made because of the site conditions, such as weather, pavement conditions, the rate of testing, and traffic control constraints.

FWD Testing

The FWD testing was performed by Dynatest and PCS/LAW, both subcontractors to ERES on this project, using a Dynatest model 8002 FWD's. To ensure the consistency in the data collected, both subcontractors used the same testing procedure and the same data reliability-check programs.

Initially, the researchers proposed that the SHRP protocol for FWD testing be adopted in this project, but for economic reasons a modified procedure was adopted. The modified procedure uses the same sensor spacings (as shown in figure 6) and drop sequence (12 kip [53 kN], 9 kip [40 kN], 12 kip [53 kN], and 16 kip [71 kN]) as the SHRP protocol, but uses a single drop at each load level, rather than the three drops required by the SHRP protocol.

The FWD testing pattern used in the field work is illustrated in figure 7. The same testing pattern was used on all sections, including those with a widened outside lane; thus, on the sections with a widened outside lane, the edge testing was performed at the paint stripe (for the lane), rather than the actual slab edge. If a tied PCC shoulder was present, additional testing was performed to determine LTE at the shoulder joint. Using the testing pattern shown in figure 6, a maximum of 10 slabs were tested at each test section, resulting in the following test locations:

- 10 midslab locations for backcalculation of concrete elastic modulus and effective *k*-values.

- 20 transverse joint locations for determination of LTE and void detection.

- 10 slab edge locations for consideration of fatigue damage and LTE for sections provided with tied PCC shoulders.

Representative slabs were selected for the FWD testing, and the 10 slabs tested were spread over the length of the section. After testing, the validity and reasonableness of the FWD data were checked using the SHRP-developed programs FWDSCAN and FWDCHECK.

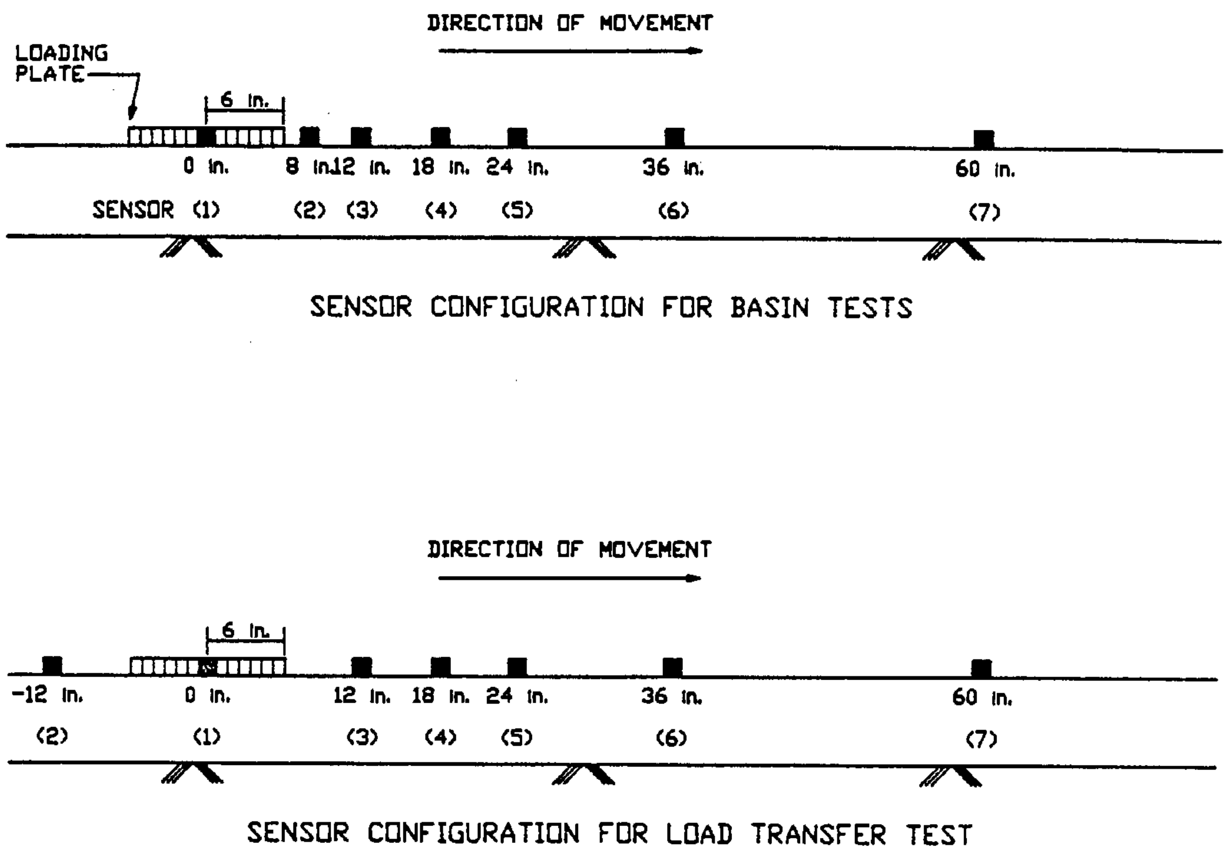


Figure 6. Sensor configurations used in the FWD testing.

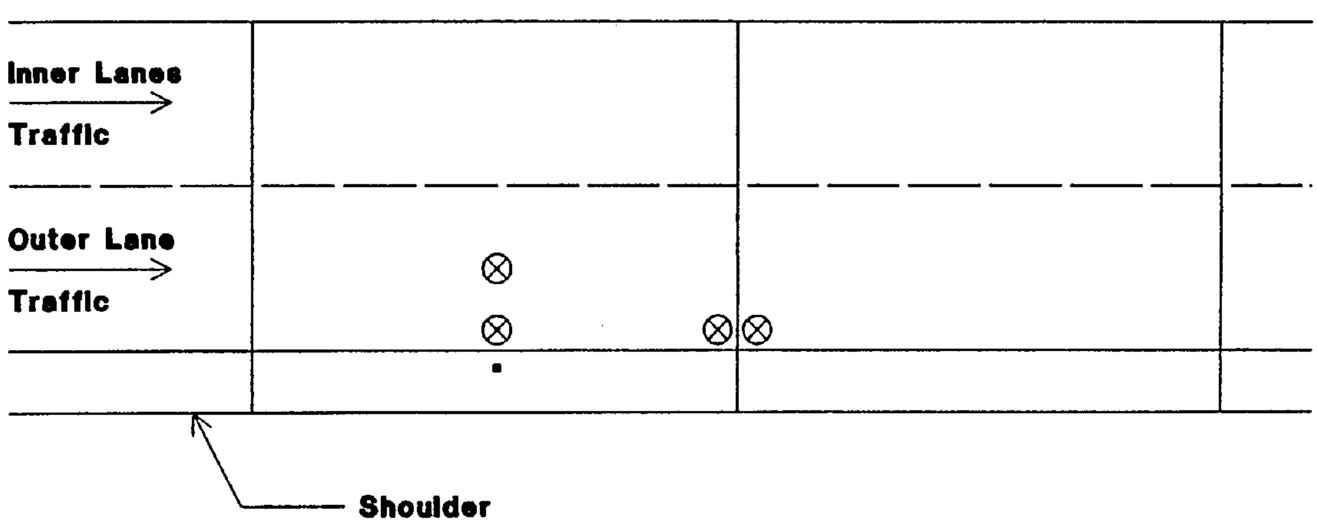


Figure 7. FWD testing pattern.

With few exceptions, FWD testing was performed while the temperature of the pavement at mid-depth was less than 80 °F (27 °C) to avoid the influence of slab expansion on the measurement of joint load transfer and void detection. The pavement temperatures were measured by drilling holes in the pavement at desired depths, dropping a small amount of mineral oil in the holes, and then measuring the temperature of the mineral oil using a temperature probe.

As shown in figure 8, three holes were drilled in the pavement to measure temperatures at the surface, mid-depth, and bottom of the PCC slab. Best efforts were made to adhere to the maximum temperature established for the testing, but exceptions had to be made on a few occasions to allow the testing to be completed in a reasonable time frame. On those few occasions, priorities were given to complete the testing of the corners (for LTE) and edges before the temperatures became excessive, because the high temperatures would most adversely affect the results of testing at those locations.

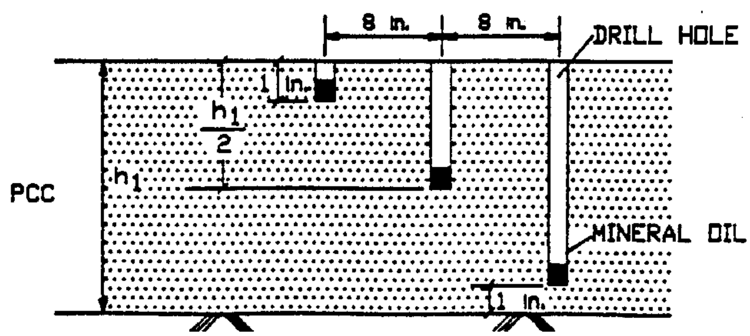


Figure 8. Arrangement of pavement temperature measurement holes.

Coring and Boring

Coring and boring were performed on most of the new sections and on some of the original 95 sections. This consisted of retrieving 6-in (152-mm) diameter cores from center locations and from locations directly over the transverse joints of selected slabs. Most of the cores taken from the original 95 sections were joint cores. The center cores were later tested in the laboratory in splitting tensile and the joint cores were examined visually. Base, subbase, and subgrade materials were retrieved from beneath some of the cores to verify the pavement structure and for use in the drainage analysis.

On this project, 6-in (152-mm) diameter cores were used rather than 4-in (102-mm) diameter cores because the strength correlations with the smaller diameter cores were shown to be adversely influenced by the aggregate size and type. If a stabilized base is present, it was also cored and tested in split tensile. Erodibility of the base was also assessed using visual procedures proposed by PIARC.[9]

The joint cores were taken about 12 in (305 mm) from the pavement edge to examine any deterioration beneath the slab corner. These cores were visually examined for subjective rating of their condition. If the pavement is doweled, attempts were made to cut the core through the outermost dowel, thus allowing the detection of any signs of "socketing" or reduced dowel bar diameter due to corrosion. The joint cores were also examined for any signs of material durability problems.

Subsurface samples were taken by boring beneath the cores on many of the sections. The samples were taken from beneath either a center or a joint core. On those sections that were bored, attempts were made to retrieve a sample from every layer, including the subgrade; however, it was not always possible to obtain these samples. On a few of the sections (the sections in FL 4, CA 9-10, and West Virginia sections) a subgrade sample could not be retrieved because of either excessive depth to the subgrade or difficulties encountered in drilling through the pavement layers.

South Dakota Profiler Testing

Profile measurements using the South Dakota profiler were taken from every section included in this study. The South Dakota Road Profiler has the advantage of not requiring the intensive calibration procedures of response-type road roughness systems and has proven to provide reliable and repeatable results. Again, the SHRP protocol for measurements was followed and the profiler was calibrated on a daily basis during testing trips throughout the project. The equipment calibration consisted of calibrating the height displacement sensors, the accelerometers, and the distance measuring sensor. The data generated by the profiler is reported in IRI units.

Both half-car and quarter-car IRI values were obtained from the profiler testing. The quarter-car IRI measures the IRI for each wheel path (left and right wheel path). The average of left and right wheel path IRI is reported in the summary tables in appendix A of Volume IV.

At each test section, two passes were made and the IRI value computed. It was planned that if the IRI values from the two passes were within 44 in/mi (0.69 m/km) of each other, then no additional passes would be made, but if the IRI values from the two passes differed by more than 44 in/mi (0.69 m/km), one additional pass would be made. The third pass was not required on any of the sections tested by this criteria, but additional passes were made whenever the operator felt that the measurement might be inaccurate due to such events as having to veer to one side or another, or not being able to maintain constant vehicle speed. A fourth pass was made on some of the sections when the IRI value from the third pass differed significantly from the first two passes. Whenever additional passes were made, measurements were taken from all of the sections within the project being profiled and the results reported.

Each section to be profiled was clearly marked on the pavement by a survey crew before the measurement took place. The field testing schedule was coordinated to ensure that a field survey crew was either at the project site with the profile crew, or preceded the profile crew. Since the profile measurements are obtained at highway speeds, this work was performed either at the end of the day, after all of the other testing had been completed and the traffic control devices removed, or at a later date.

Laboratory Testing

The laboratory testing for this project consisted of split tensile testing of the center-cores and characterization of the subsurface material. The splitting tensile strength testing on the center cores was performed in accordance with AASHTO T-198 (ASTM C 496). Based on the results of split tensile testing, concrete modulus of rupture was estimated using Hammit's equation:[10]

$$M_R = 1.02 \times f_t + 210 \tag{1}$$

where:

M_R = concrete modulus of rupture, lb/in^2.
f_t = split tensile strength, lb/in^2.

A number of tests were performed on the subsurface layer samples retrieved by boring. These include gradation, liquid limit, plastic limit, specific gravity, and moisture content. The AASHTO Classification was also determined for both the granular materials and the subgrade soil. This information, along with the layer thicknesses obtained by measuring the layer samples, was used to verify the pavement structure. All of this information will be used later in the drainage evaluation. The results of the laboratory testing are summarized in appendix A of Volume IV.

Data Reduction Procedures

A summary of all of the data collected under this project is presented in appendix A of Volume IV. Efforts were made to keep the data elements in the summary tables consistent with those in the summary tables from the previous study so that direct comparisons could be made; however, some deviations were unavoidable. Whereas Mays Meter was used to measure roughness in the previous study, an SD Dakota Profiler was used in this study, and the results were reported in IRI units. Some changes were also made to improve the clarity of the data presented, but in those instances, either the original data were converted to the new format, or the new data were presented in both the new and old format. Examples of these are:

- The longitudinal cracking from both lanes were combined and reported in terms of LF/mi, since this is the way the data will be used in the analysis. In the previous report, longitudinal cracking in each lane were reported separately (although the data were not used this way in the analysis).

- In addition to the average load transfer efficiency (LTE), the approach-side and leave-side LTE were reported separately. Only the average LTE was reported in the previous report.

In the following sections, the procedures used to organize all of the data collected under this project are described.

Manual Condition Survey Data

The data collected from the field were entered into a computer data base and summarized into a format that can be easily manipulated in the analysis work and in creating the SHRP LTPP-type data base (an ORACLE data base). The data were first entered into spreadsheet files, one file per section. All of the data on the field survey sheets (figures 2, 3, and 4), including the data extracted from the distress maps, were transferred to the individual spreadsheet files; hence, a very detailed data base was created that contains distress data for individual slabs. Simple data reduction that could be performed on the raw data (such as determining the average values, maximum values, minimum values, percentages, and normalizing distress quantities) was performed on the individual spreadsheets. The summary information from each file was then aggregated into a large flat file that consists of rows of sections and columns of reduced performance data (i.e., each row in the flat file contains all of the reduced performance data for a pavement section). The pavement performance data shown in the summary tables were extracted from the flat table.

Automated Distress Survey Data

The data from the automated distress survey were summarized at three different levels of detail for each section:

- Distress data summary for the entire experimental section.

- Distress data summary for each 1-mi (1.6-km) segment, starting at the beginning of the section (for long sections).

- Distress data summary for each 0.1-mi (0.16-km) segment, starting at the beginning of the section.

The distress data in each summary are presented in terms of the total quantity of each type of distress at each severity level (high, medium, low) in the following units:

- Number. Corner breaks, transverse cracking, blowups, popouts, AC patch/patch deterioration, PCC patch/patch deterioration.

- Number of affected joints. D-cracking, joint seal damage (transverse joint), spalling (transverse joint), water bleeding and pumping.

- Linear feet of affected joint. Joint seal damage, spalling of longitudinal joint, spalling of transverse joint, lane-to-shoulder joint separation.

- Linear feet. Longitudinal cracking, transverse cracking.

- Area. D-cracking, scaling, AC patch/patch deterioration, PCC patch/patch deterioration.

In addition to these distress quantities, the section length surveyed and the number of joints are also reported so that these distress quantities could be normalized. Also, a complete distress map of all of the sections surveyed was generated.

Most of the pavement sections selected for this study were surveyed by both automated and manual methods. However, the two sets of data are not completely redundant, because the boundaries of the automated survey sections do not coincide with those of the manual survey. This is not a serious drawback, because a reliable set of distress data have been obtained for most sections by manual means. Any gaps in the manually collected data will be supplemented by the data obtained from the automated survey.

The automated survey was typically conducted over the entire length of the experimental pavements, and the survey generated an abundant amount of valuable, additional distress data. These data were used in numerous ways in this project, including the following:

- To supplement the manually collected data in the development of performance prediction models. Several sections in Minnesota and in Wisconsin were surveyed using the automated method only. Also, many replicate sections were included in the automated survey.

- To validate the performance models. The numerous replicate sections included in the automated survey provide an independent set of data that could be used to test the performance models developed under this project.

- To evaluate the sampling accuracy of selected projects. The automated surveys were conducted over the entire length of the experimental sections, whereas the manual survey was conducted over a relatively short sample.

The data from the automated survey were entered into a computer data base of its own to ensure ready access to the data for any interested parties. Of the three types of summaries—summary by project, 1-mi (1.6-km) segment, and 0.1-mi (0.16 km) segment—only the summaries by project and 0.1-mi (0.16-km) segment were entered into the data base. The summary by 1-mi (1.6-km) segment were not included, because this information can be derived from the summary by 0.1-mi (0.16-km) segment and many sections are shorter than 1 mi (1.6 km).

The data base containing the data from the automated distress surveys is simply a collection of spreadsheet files consisting of the following:

- One file for the project summaries—the individual project summary files provided by PASCO (comma separated variables) were aggregated into a single spreadsheet, and the following information was added:

    - Column headings
    - Project location information
    - Project ID's.

- One file per State for the summaries by 0.1-mi (0.16-km) segments.

The spreadsheet data base was created, rather than including the automated survey data in the SHRP LTPP (ORACLE) data base, because it offers several advantages. A simple procedure is available for entering the automated survey data into the SHRP LTPP data base; however, the data base assumes that any data entered in the data base for the sections identified in the data base are obtained over the exact boundaries of those sections. The discrepancies in the section boundaries between the automated and the manual survey sections pose difficulties in this regard, but a more serious problem is that the constraint imposed by the LTPP data base precludes the possibilities of entering the detailed distress data from the automated survey (the summaries by 0.1-mi [0.16-km] segments). The use of the spreadsheet data base allows all of the data collected by automated survey to be preserved and offers ready access to the data. In addition, this approach eliminates the need to reconcile the differences in the section boundaries between the automated and manual survey sections, which can introduce arbitrary errors.

FWD Testing Data

The FWD data from the field testing was used to determine the subgrade modulus of reaction, *k*, elastic modulus of concrete, E_c, and load transfer efficiency. The backcalculation scheme developed for SHRP P-020, *Long-Term Pavement Performance Data Analysis*, was used to determine the subgrade *k* and the concrete modulus, E_c.[11] This procedure combines the theoretical solutions and the results of dimensional and statistical analyses to allow direct solution of the *k* and E_c values from the FWD data.

Backcalculation

The basic approach in backcalculating the pavement system parameters from the FWD testing results is to compare the measured deflection values from the FWD testing to those predicted by plate theory and determine the values of E_c and *k* that, when used in the equations given by the theory, will produce the measured values. In this work, the PCC deflections are usually represented by a single, normalized parameter, AREA, that uniquely characterizes the deflected shape of the slab. The stiffness of the slab-foundation system is characterized by the radius of relative stiffness, ℓ_k, which combines E_c and k. The AREA is defined as the cross-sectional area of the deflection basin between the center of the load plate and the outermost sensor used in the backcalculation, approximated by the trapezoidal rule, and computed with all deflections normalized to the maximum deflection, d_o. For the SHRP sensor arrangement shown in figure 6 (sensors at 0, 8, 12, 18, 24, 36, and 60 in [0, 203, 305, 457, 610, 914, 1,524 mm]), the expression for the AREA is as follows:

$$AREA = 4 + 6\frac{d_8}{d_0} + 5\frac{d_{12}}{d_0} + 6\frac{d_{18}}{d_0} + 9\frac{d_{24}}{d_0} + 18\frac{d_{36}}{d_0} + 12\frac{d_{60}}{d_0} \tag{2}$$

where d_i is the deflection at sensor i in inches. The deflection values are normalized to d_o to remove the dependence of the AREA on the load levels; thus, the AREA has the unit of length. The radius of relative stiffness (ℓ_k) for slabs on a dense liquid foundation is as follows:

$$\ell_k = \sqrt[4]{\frac{E_c h^3}{12(1-\mu_c^2)k}} \tag{3}$$

where:

ℓ_k = radius of relative stiffness, in.
E_c = PCC elastic modulus, lb/in^2.
h = slab thickness, in.
μ_c = PCC Poisson's ratio (typically 0.15).
k = subgrade modulus of reaction, lb/in^2/in.

The backcalculation scheme used in this project is based on the fact that if the load radius is fixed (as in the case of FWD testing), a unique relationship exists between the normalized cross sectional area of the deflection basin, AREA, and the radius of relative stiffness, ℓ_k. To allow direct solution of ℓ_k, given the AREA, the closed form equation for deflection of a PCC slab on a dense liquid foundation given by Losberg was solved for a range of ℓ_k values (from 18 to 80 in [457 to 2,032 mm]) and the following regression equation was developed:[11]

$$\ell_k = \left[\frac{\ln\left(\frac{k_1 - AREA}{k_2}\right)}{-k_3}\right]^{1/k_4} \tag{4}$$

where:

k_1 = 60.
k_2 = 289.708.
k_3 = 0.698.
$1/k_4$ = 2.566.

The R^2 for equation 4 is 99.99 and the standard error of the estimate (σ_Y) is 0.178. It is important to note that equation 4 is valid only for an 11.8-in (300-mm) diameter load plate (load radius, a = 5.9055 in [150 mm]) and the AREA obtained using equation 2.

Equation 4 allows ℓ_k to be determined directly from the FWD testing results; however, ℓ_k is a composite parameter that represents the combined stiffness of the slab and the subgrade. In order to resolve the subgrade k and E_c from ℓ_k, another relationship is needed. The following relationship between the nondimensional deflection coefficient, d_r^*, and the pavement system parameters was used to determine k and E_c from ℓ_k:

$$d_r^* = \frac{d_r k \ell_k^2}{P} = \frac{d_r D}{P \ell_k^2} \tag{5}$$

where:

d^*_r = nondimensional deflection coefficient
d_r = deflection at the surface of a slab at radial distance r from the load, in.
k = subgrade modulus of reaction, lb/in²/in.
ℓ_k = radius of relative stiffness, in.
P = applied load, lb.
D = flexural rigidity, lb-in.

and where:

$$D = \frac{E_c h^3}{12(1 - \mu_c^2)} \tag{6}$$

The nondimensional deflection coefficient is a function of the load radius, radial distance from the load, and ℓ_k. If the load radius is fixed (as in the case of FWD testing), then the deflection coefficient is a function only of radial distance from the load and ℓ_k. The same data used to develop the regression equation for ℓ_k was used to develop a regression equation for d_r^*:

$$d_r^* = a e^{[-b e^{(-c \ell_k)}]} \tag{7}$$

The regression coefficients *a, b,* and *c* are given in table 3. By rearranging equation 5, the expression for *k* can be obtained:

$$k = \frac{P d_r^*}{d_r \ell_k^2} \tag{8}$$

Once the ℓ_k and d_r^* have been determined from AREA using equations 4 and 7, the *k* value may be estimated from any of the measured sensor deflections (d_r) using equation 8. The *k* value for the entire deflection basin can then be taken as the average of the *k* values given by the individual sensor deflections.

Table 3. Regression coefficients for d_r^* versus ℓ_k relationships.

Radial Distance, r (in)	a	b	c
0	0.12450	0.14707	0.07565
8	0.12323	0.46911	0.07209
12	0.12188	0.79432	0.07074
18	0.11933	1.38363	0.06909
24	0.11634	2.06115	0.06775
36	0.10960	3.62187	0.06568
48	0.10241	5.41549	0.06402
60	0.09521	7.41241	0.06255

Notes: $R^2 \geq 99.7$ percent for all models.
$\sigma_Y \leq 0.001$ for all models.

The expression for determining E_c based on each sensor deflection can be obtained by rearranging equations 5 and 6:

$$E_c = \frac{12(1-\mu_c^2) P \ell_k^2 d_r^*}{d_r h^3} \tag{9}$$

As with the *k* value, the E_c for the entire deflection basin can be obtained by averaging the estimate of E_c given by each sensor deflection.

The backcalculation process described in this section can be summarized as follows:

- Compute an AREA from the measured deflections.
- Determine ℓ_k using equation 5.
- Determine nondimensional deflection coefficients (d_r^*) at each sensor using equation 7.
- Determine the *k* values based on each sensor deflection using equation 8, and then average the values to obtain the *k* value for the entire deflection basin.
- Determine E_c based on each sensor deflection using equation 9, and then average the values to obtain the E_c for the entire deflection basin.

A computer program that automates this procedure was used to backcalculate the values of subgrade *k* and E_c. The backcalculation results are given in appendix A of Volume IV.

Load Transfer Efficiency

The deflection LTE at the PCC pavement joints and cracks is defined as the ratio of the deflection of the unloaded side to that of the loaded side:[12]

$$LTE = \frac{\delta_U}{\delta_L} * 100 \qquad (10)$$

where:

LTE = load transfer efficiency, percent.
δ_U = deflection of the unloaded side.
δ_L = deflection of the loaded side.

The LTE given by equation 10 is valid if the deflection measurements were taken at the points immediately to either side of the joint or crack; however, because of equipment limitations, this is not possible. In reality, the measurements are taken 6 in (152 mm) away from the joint, resulting in the ability of slab bending over the 12-in (305-mm) separation to affect the measurements. To compensate for the slab bending, a correction factor is applied to equation 10:

$$LTE = \frac{\delta_U}{\delta_L} * B * 100 \qquad (11)$$

where B is the ratio of the deflection at the load plate to the deflection 12 in (305 mm) away from the load plate under center loading condition (basin testing).

The LTE values determined from the FWD testing data are given in appendix A of Volume IV. The column labeled "Approach" refers to the results from testing conducted with the load placed at the outer approach corner of the slab, and the column labeled "Leave" refers to the results from testing conducted with the load plate placed at the outer leave corner of the slab.

Void Detection

The FWD data were also used to determine the extent of loss of support at the slab corners. The loss of materials from beneath the transverse joints, especially near the slab corners, can lead to faulting and corner breaks. This is an important design consideration to prevent premature loss of serviceability.

The loss of support at the slab corners can be determined by examining the corner deflections from the FWD testing conducted at various load levels. The step-by-step procedure for void detection presented in reference 13 is summarized below:

1. Measure deflection at the slab corner under a range of load levels that includes a 9,000-lb (40-kN) load. The SHRP load sequence used in FWD testing—9 kip (40 kN), 12 kip (53 kN), and 16 kip (71 kN)—satisfies this requirement. The testing should be conducted in the temperature range between 50 °F (10 °C) to 80 °F (27 °C).

2. Plot the results on a load (x-axis) versus deflection (y-axis) graph and draw a best fit line through the points. Extend this line down to determine the y-intercept.

3. A corner that has full support will show the y-intercept very close to zero. Any line that shows the y-intercept greater than 0.003 in (0.076 mm) is considered an indication of loss of support.

The concept behind this procedure is simple: if a void exists under a slab, the slab will exhibit a non-linear response and show a higher deflection at lower load levels, causing the y-intercept to plot higher in the graph. In general, a larger y-intercept may be expected for larger voids.

The loss of support is expressed as the percent of corners with voids. The percent corners with voids is taken as the fraction of the corners with voids, as determined using the above procedure, among the corners tested. The results of this analysis showed that the testing conducted with the load plate placed on the leave slab indicates a greater percentage of the corners with voids than the testing conducted with the load plate placed on the approach slab. This result is expected, since voids typically form under the leave side of a joint. The results of the void evaluation are given in appendix A of Volume IV. The percent corners with voids given in appendix A of Volume IV are the results of the testing conducted with the load plate placed on the leave slab.

Profile Data

The profile data from the field testing were reduced to IRI by section and are reported in appendix A of Volume IV. Both the quarter-car IRI (for each wheel path) and half-car IRI were determined. The IRI values reported in the appendix are the average of the left wheel path and right wheel path IRIs.

Data Base

Virtually all of the data collected under this project have been entered into computer files. All of the data in these files have been validated, and they have been entered into the ORACLE data base in the LTPP format (all, except the automated survey data), which is a comprehensive data base that was designed by SHRP to track pavement performance in the LTPP program. The data elements for this data base include inventory data, traffic data, environmental data, performance data, as well as testing results. The LTPP data base was designed to store the data in their most basic format. For example, in the case of the FWD testing data, the individual sensor deflections, load, and testing temperatures are stored in the data base, rather than the backcalculated results and the calculated LTE values. In the case of performance data such as faulting, the individual faulting measurements and the location of the measurement are stored, rather than average faulting. The intent of the LTPP data base is to provide the raw performance data rather than any interpretation of the data.

As described in the *Data Reduction* section, the types of detailed performance data required by the LTPP data base have already been entered into spreadsheet files. A number of utilities has been developed under SHRP LTPP for importing spreadsheet files into the LTPP data base. Once the data in the spreadsheet files have been validated, they will be aggregated into a number of tables that are convenient for moving the data into the LTPP data base and then entered into the LTPP data base using the SHRP utilities.

3. PERFORMANCE OF PAVEMENT SECTIONS

Introduction

During the summer of 1992, over 270 concrete pavement sections throughout the United States (and several in Canada) were manually surveyed and inspected. These pavement sections were either single sections containing unique design features (e.g., widened traffic lane), or part of an experimental project constructed by a State Highway Agency (SHA) to evaluate the performance of different concrete pavement designs and design features. The selection of the sections was primarily based on a 1987 FHWA study that evaluated the performance of 95 concrete pavement sections. The intent of this study is to re-evaluate those 95 sections so that performance trends and deterioration rates can be identified. Furthermore, additional sections can be added to expand the range of design features in the study and thereby increase the validity of the analysis. Many of the sections added to the study were part of the 1987 projects, but not surveyed at that time; the rest of the additional sections came from newly identified experimental projects. While the focus of the investigation was on jointed concrete pavements, several of the experimental projects contained adjacent CRCP sections that were also included in the evaluation.

Each of the 270 sections was subjected to a condition survey and drainage evaluation, generally over a length of about 1000 ft (305 m) (or the actual length of the sections, whichever was shorter). In addition, many of these sections were tested using the falling weight deflectometer. Coring and subsurface boring operations were also conducted on selected sections. As previously described, 95 of the 270 sections surveyed in 1992 had been surveyed in 1987 using similar procedures.

To supplement the field testing, distress data were also collected using automated photographic procedures. These data were collected in 1991 and 1992 on the larger, multi-section projects included in the study. Whereas the manual surveys were conducted on short sample units, the automated survey procedures performed virtually 100 percent sampling. Automated distress surveys were conducted on additional sections within a project that were not included in the manual surveys, including replicate sections. However, the automated procedures were not able to provide faulting measurements of transverse joints. A summary of the number of sections surveyed by the different methods is given in table 4. Many of the surveys done using automated methods were conducted on the same design sections as the manual procedures.

This chapter presents a summary of the performance of the sections that were included in the field testing and evaluation; data are presented for a total of 308 pavement sections. The location of each project is given, the design variables of each individual sections are described, and a summary of the results from the field testing

Table 4. Summary of sections surveyed in 1987 and 1992.

		NUMBER OF SECTIONS INCLUDED IN:		
		1987 Manual Surveys	1992 Manual Surveys	1991/1992 Automated Surveys (number of same sections also included in manual surveys)
DRY-FREEZE	JPCP	3	11	26 (10)
	JRCP	17	29	26 (26)
	CRCP	0	0	0 (0)
DRY-NONFREEZE	JPCP	17	30	23 (23)
	JRCP	0	0	0 (0)
	CRCP	0	6	6 (6)
WET-FREEZE	JPCP	20	61	63 (58)
	JRCP	24	78	62 (62)
	CRCP	0	10	7 (7)
WET-NONFREEZE	JPCP	13	43	28 (28)
	JRCP	1	1	1 (1)
	CRCP	0	1	1 (1)
TOTAL	JPCP	53	145	140 (119)
	JRCP	42	108	89 (89)
	CRCP	0	17	14 (14)
GRAND TOTAL		95	270	243* (222)

* Includes replicate sections only (not additional sections).

is presented. Detailed design and performance information for each section, including 1987 performance data (where applicable), is found in appendix A of Volume IV.

The performance of the pavement sections is expressed for the outer traffic lane in terms of average transverse joint faulting, deteriorated transverse cracks per mile (all severity levels for JPCP and medium- and high-severity for JRCP), percent slabs with transverse cracks (all severity levels) for JPCP, feet of longitudinal cracking (all severity levels) per mile, percent of joints exhibiting joint spalling (medium- and high-severity), present serviceability rating (PSR), and International Roughness Index (IRI). To assist in evaluating the performance of the pavement sections, table 5 provides critical levels of deterioration—values at which the pavement is considered to be in need of some sort of rehabilitation—for each distress.

Table 5. Critical values for key performance indicators.

Performance Indicator	JPCP	JRCP
Joint Faulting	0.13 in	0.26 in
Transverse Cracking	10% slabs cracked, or 70 cracks/mi	70 deteriorated (M-H) cracks/mi
Longitudinal Cracking	500 ft/mi (all levels)	500 ft/mi (all levels)
Joint Spalling	15-20% joints spalled, or 50 spalls/mi	20-30% joints spalled, or 25 spalls/mi
PSR	3.0-3.5	3.0-3.5
IRI	125-175 in/mi	125-175 in/mi

1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km

Performance Summary of Projects in Dry-Freeze Zones

Projects surveyed in the dry-freeze zone are located only in the State of Minnesota. While Minnesota is actually included in a transition region between dry-freeze and wet-freeze, it is classified as being in the dry-freeze region because it is certainly drier than States in the wet-freeze region. For the sections in Minnesota, the Corps of Engineers (COE) Freezing Index ranges from 1688 to 2188 °F-days (937.8 to 1215.6 °C-days), and the Thornthwaite Moisture Index ranges from 0 to 10. The annual precipitation for the region ranges from 23 to 30 in (584 to 762 mm).

<u>Minnesota 1 and Minnesota 5 (I-94, Rothsay)</u>

Minnesota 1, the experimental project on I-94 near Rothsay, Minnesota, was constructed in 1970 to evaluate the effect of base type, slab thickness, and load transfer on jointed reinforced concrete pavement performance.[14,15] The variables include aggregate (AGG), asphalt-treated (ATB, 5 percent of AC-10), and cement-treated (CTB, 5 percent cement) bases; 8- and 9-in (203- and 229-mm) JRCP slabs (0.09 and 0.08 percent steel, respectively); and doweled (1-in [25-mm] diameter bars) and nondoweled joints. Each design section rests on an A-6 subgrade. The design matrix for this project is shown in table 6.

Sections 1-1 through 1-12 (westbound direction) were surveyed in both 1987 and 1992, while sections 1-13 through 1-24 (eastbound direction) were surveyed in 1992 only. In 1984, sections 1-1, 1-3, 1-9, and 1-11 in the westbound direction and sections 1-17, 1-19, 1-21, and 1-23 in the eastbound direction were diamond ground (truck lanes only) and retrofitted with tied and doweled edge beams.

Table 6. Experimental design matrix for MN 1 and MN 5.

		8-in (203-mm) JRCP (0.09% Steel)		9-in (229-mm) JRCP (0.08% Steel)	
		No Dowels	1-in (25-mm) Dowels	No Dowels	1-in (25-mm) Dowels
Constructed 1970 27-ft (8.2-m) Skewed Joints	6-in (152-mm) AGG	MN 1-3* MN 1-23*	MN 1-4 MN 1-24	MN 1-1* MN 1-21*	MN 1-2 MN 1-22
	5-in (127-mm) ATB (5% AC-10)	MN 1-5 MN 1-15	MN 1-6 MN 1-16	MN 1-7 MN 1-13	MN 1-8 MN 1-14
	5-in (127-mm) CTB (5% cement)	MN 1-11* MN 1-17*	MN 1-12 MN 1-18	MN 1-9* MN 1-19*	MN 1-10 MN 1-20
Constructed 1969 39-ft (11.8-m) Joints	3-in (76-mm) AGG				MN 5 (0.04% Steel)

* Sections were diamond ground and edge beam (tied and doweled) was added in 1984.
Sections 1-1 through 1-12 were also surveyed in 1987.
No subbase included in any section.

Minnesota 5, a pavement section located near the MN 1 project, is included in the study because it is typical of Minnesota's concrete pavement design of the 1960's. This section was built in 1969 and consists of a 9-in (229-mm) thick JRCP (0.04 percent steel) with doweled transverse joints spaced at 39-ft (11.8-m) intervals. It has a 3-in (76-mm) thick aggregate base and rests on an AASHTO A-6 subgrade.

Traffic Loadings

This four-lane divided roadway is currently subjected to a two-way Average Daily Traffic (ADT) of 6,000 vehicles per day, including 22 percent heavy trucks (FHWA classification 5 and greater). In 1987, it had sustained approximately 5.5 million 18-kip (80-kN) equivalent single-axle load (ESAL) applications in the outer traffic lane, and through 1992 had received approximately 7.4 million ESAL applications.

Performance

Table 7 contains a summary of the performance data for MN 1 and MN 5. Joint faulting, deteriorated (medium- and high-severity) transverse cracks per mile, longitudinal cracking per mile (all severity levels), percent of joints spalled (medium and high severity), PSR values, and IRI values are given for the outer lane of each section. Performance data are provided for the 1987 surveys, for the 1992 reinspection of the 1987 sections, and for the 1992 replicate sections not surveyed in 1987. These performance data are also presented graphically in figure 9.

Table 7. Summary of 1987 and 1992 outer lane performance data for MN 1 and MN 5.

			8-in (203-mm) JRCP (0.09% steel)						9-in (229-mm) JRCP (0.08% steel)						Average		
			No Dowels			1-in (25-mm) Dowels			No Dowels			1-in (25-mm) Dowels					
			1987 Survey	1992 Survey	1992 Rep.	1987 Survey	1992 Survey	1992 Rep.	1987 Survey	1992 Survey	1992 Rep.	1987 Survey	1992 Survey	1992 Rep.	1987 Survey	1992 Survey	1992 Rep.
Age at Survey, years			17	22	22	17	22	22	17	22	22	17	22	22	17	22	22
ESAL's, millions			5.5	7.4	7.4	5.5	7.4	7.4	5.5	7.4	7.4	5.5	7.4	7.4	5.5	7.4	7.4
27-ft (8.2-m) Skewed Joints	6-in (152-mm) AGG		1-3	1-3	1-23	1-4	1-4	1-24	1-1	1-1	1-21	1-2	1-2	1-22			
		Faulting, in	0.12	0.17	0.15	0.08	0.11	0.09	0.07	0.13	0.19	0.10	0.15	0.12	0.09	0.14	0.14
		Det. Tr. Crks/mi	33	49	73	47	83	86	0	0	0	23	45	23	26	45	48
		Long. Cracks, ft/mi	301	350	220	0	0	383	0	0	0	0	0	0	75	88	151
		% Joints Spalled	8	56	28	15	48	38	4	68	16	14	61	50	10	58	33
		PSR	3.4	3.4	3.0	3.3	3.3	3.2	3.7	3.3	3.1	3.3	3.5	3.2	3.4	3.4	3.1
		IRI, in/mi	—	161	126	—	105	140	—	156	152	—	179	143	—	150	140
	5-in (127-mm) ATB (5% AC-10)		1-5	1-5	1-15	1-6	1-6	1-16	1-7	1-7	1-13	1-8	1-8	1-14			
		Faulting, in	0.11	0.19	0.21	0.08	0.08	0.14	0.05	0.11	0.17	0.09	0.10	0.08	0.08	0.12	0.15
		Det. Tr. Crks/mi	41	73	106	0	8	49	0	16	65	70	70	110	28	42	82
		Long. Cracks, ft/mi	2656	2909	3724	3371	3293	3455	2958	4164	5997	5233	5921	6234	3554	4071	4853
		% Joints Spalled	24	84	92	31	96	92	15	65	92	69	96	96	35	85	93
		PSR	3.4	3.5	3.0	3.4	3.1	3.3	3.6	3.6	3.1	3.4	3.8	3.1	3.4	3.5	3.1
		IRI, in/mi	—	186	151	—	186	149	—	174	128	—	109	166	—	164	148
	5-in (127-mm) CTB (5% cement)		1-11	1-11	1-17	1-12	1-12	1-18	1-9	1-9	1-19	1-10	1-10	1-20			
		Faulting, in	0.11	0.16	0.25	0.10	0.12	0.15	0.08	0.11	0.12	0.08	0.12	0.15	0.09	0.13	0.17
		Det. Tr. Crks/mi	0	8	8	48	81	8	0	0	8	18	18	39	16	27	16
		Long. Cracks, ft/mi	4260	5374	130	1019	1255	0	0	0	722	0	0	0	1320	1657	213
		% Joints Spalled	37	77	20	29	80	52	4	38	69	6	76	31	19	68	43
		PSR	3.7	3.5	3.2	3.5	3.5	3.2	3.7	3.6	3.5	3.3	3.5	3.1	3.6	3.5	3.2
		IRI, in/mi	—	125	164	—	124	161	—	155	127	—	117	167	—	130	154
39-ft (11.8-m) Joints	3-in (76-mm) AGG											MN 5*	MN 5*				
		Faulting, in										0.09	0.13				
		Det. Tr. Crks/mi										53	79				
		Long. Cracks, ft/mi										1261	2046				
		% Joints Spalled										36	95				
		PSR										3.3	3.5				
		IRI, in/mi										—	139				
Average (not including MN 5)		Faulting, in	0.11	0.17	0.20	0.09	0.10	0.13	0.07	0.12	0.16	0.09	0.12	0.12			
		Det. Tr. Crks/mi	25	43	62	47	57	48	0	5	24	37	44	57			
		Long. Cracks, ft/mi	2406	2878	1358	1463	1516	1279	986	1388	2240	1744	1974	2078			
		% Joints Spalled	23	72	47	25	75	61	8	57	59	30	78	59			
		PSR	3.5	3.5	3.1	3.4	3.3	3.2	3.7	3.5	3.2	3.3	3.6	3.1			
		IRI, in/mi	—	157	147	—	138	150	—	162	136	—	135	159			

Sections 1-1, 1-3, 1-9, 1-11, 1-17, 1-19, 1-21, and 1-23 were diamond ground and edge beam added in 1984.
Sections 1-5, 1-7, 1-13, and 1-15 were diamond ground in 1984.
No subbase included in any section.
* MN 5 has 0.04% steel.

1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km

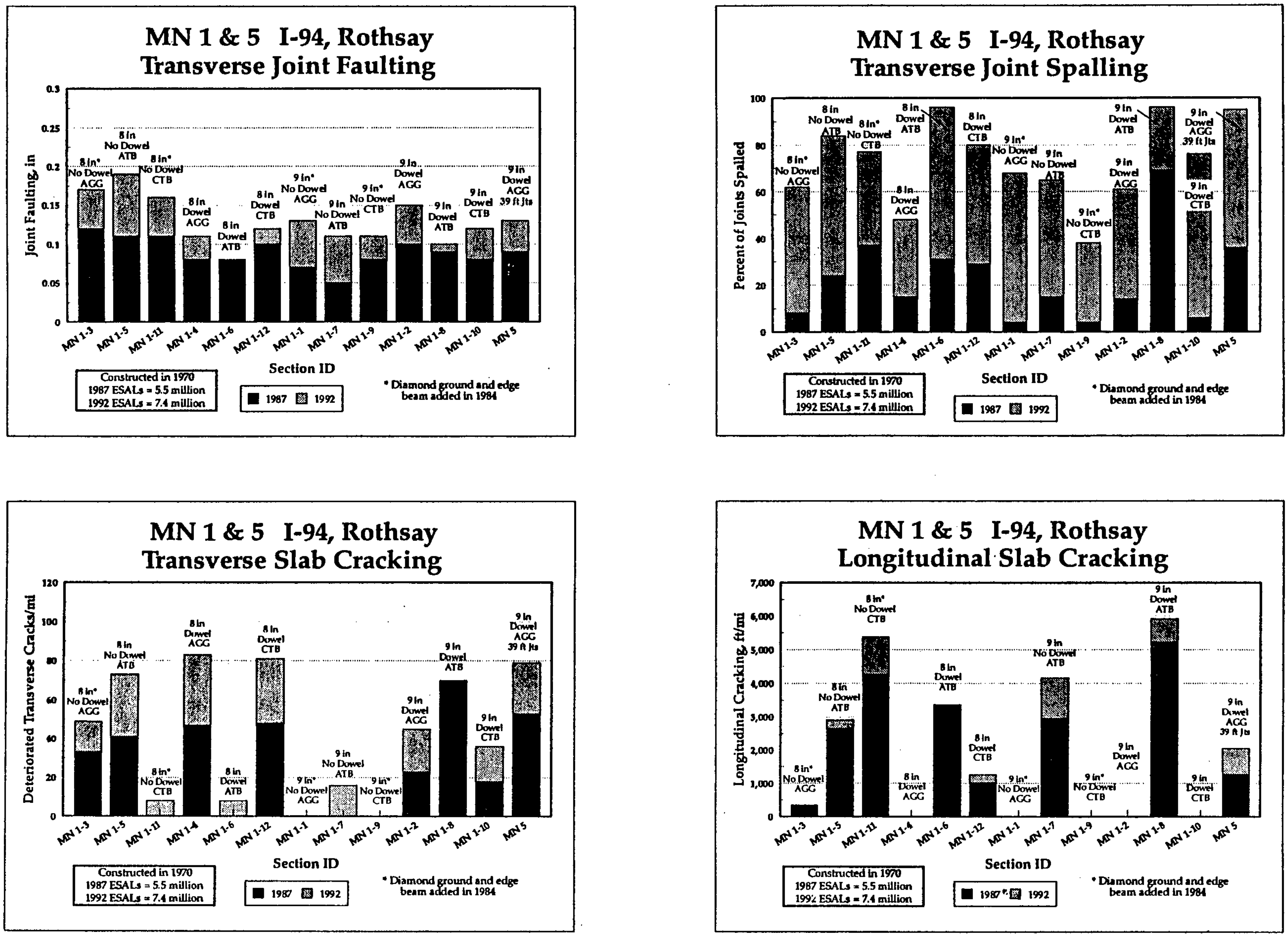


Figure 9. Performance summary for MN 1 & 5.

The 1992 data indicate, as did the 1987 data, that the sections constructed on the ATB developed a significant amount of longitudinal cracking, while the sections constructed on the AGG base display very little longitudinal cracking. This finding is confirmed by the replicate sections surveyed in the eastbound direction in 1992. In addition, nearly 90 percent of the joints on the sections constructed on the ATB exhibit joint spalling, whereas less than 50 percent of the joints on the CTB or on the AGG sections exhibit joint spalling. However, for a given method of load transfer, there are no discernible differences in joint faulting between the sections constructed on the different base types.

Several factors are believed to have contributed to the extensive longitudinal cracking that developed on the treated bases. Cracks occurred on the pavement sections constructed on CTB immediately after construction and were attributed to reflective cracks from the base.[15] Furthermore, the longitudinal joint between lanes was formed with a plastic insert, which apparently did not establish an effective weakened plane. With large curling stresses induced in the sections on treated bases, longitudinal cracking developed at locations other than the weakened plane centerline joint.

It appears from table 7 that the 1-in (25-mm) diameter dowel bars were effective in reducing joint faulting, particularly for the 8-in (203-mm) slabs. The faulting on the nondoweled sections constructed on AGG and CTB, in which the truck lanes were diamond ground and tied and doweled edge beams were added in 1984, has redeveloped to levels greater than that of the doweled sections.

However, several of the doweled sections developed significant faulting, which is attributed to the small diameter of the dowel bars. Furthermore, many of the doweled sections exhibit more transverse cracking and joint spalling than the nondoweled sections, as shown in table 7 and figure 9. This phenomenon is probably due to corrosion of the noncoated dowel bars, which can lead to the transverse joint locking up, which creates tremendous forces in the slab that either spall the joint or open mid-slab cracks.

The effect of slab thickness is not readily apparent from an examination of table 7 or figure 9. The increased slab thickness appears to reduce spalling, faulting, and cracking for the nondoweled pavement sections, although it does not appear to provide those benefits for the doweled sections. This again suggests that the dowels may be contributing to joint spalling and crack deterioration, perhaps due to their corroding and locking up the joint.

In comparing the performance of MN 1-2 (and MN 1-22) with MN 5, the effect of joint spacing on performance can be observed. MN 5 (39-ft [11.8-m] joint spacing), displayed considerably more deteriorated transverse cracks and joint spalling than its 27-ft (8.2-m) counterparts. The relative level of faulting for the sections, however, is about the same.

Roughness and Serviceability

Pavement roughness and serviceability for these sections are given in figure 10. This figure shows that most of the sections are of about the same smoothness and serviceability level. Many of these are at or near critical levels in terms of IRI. Notable exceptions are MN 1-5, MN 1-4, MN 1-12, MN 1-8, and MN 1-10, all of which (except MN 1-5) contain dowel bars.

5-Year Performance Trends

The performance of the sections from the 1987 surveys to the 1992 surveys follows a logical trend in that increased levels of deterioration are observed. Figure 11 provides 5-year performance trends for joint faulting and transverse cracking, broken out by 8- and 9-in (203- and 229-mm) slabs. This figure shows that faulting on doweled sections is increasing at a lower rate than faulting for nondoweled sections, especially for the 8-in (203-mm) slabs. In addition, the sections constructed on treated bases, particularly those constructed on ATB, appear to offer more resistance to faulting increases than the sections constructed on aggregate bases. However, all nondoweled sections, except those placed on ATB, were diamond ground in 1984.

Figure 11 also shows that transverse cracking is increasing at a more rapid rate on the 8-in (203-mm) slabs than on the 9-in (229-mm) slabs. This means that more cracks are breaking down and deteriorating on the thinner slab sections. Cracking generally is increasing more rapidly on the doweled sections than on the nondoweled sections.

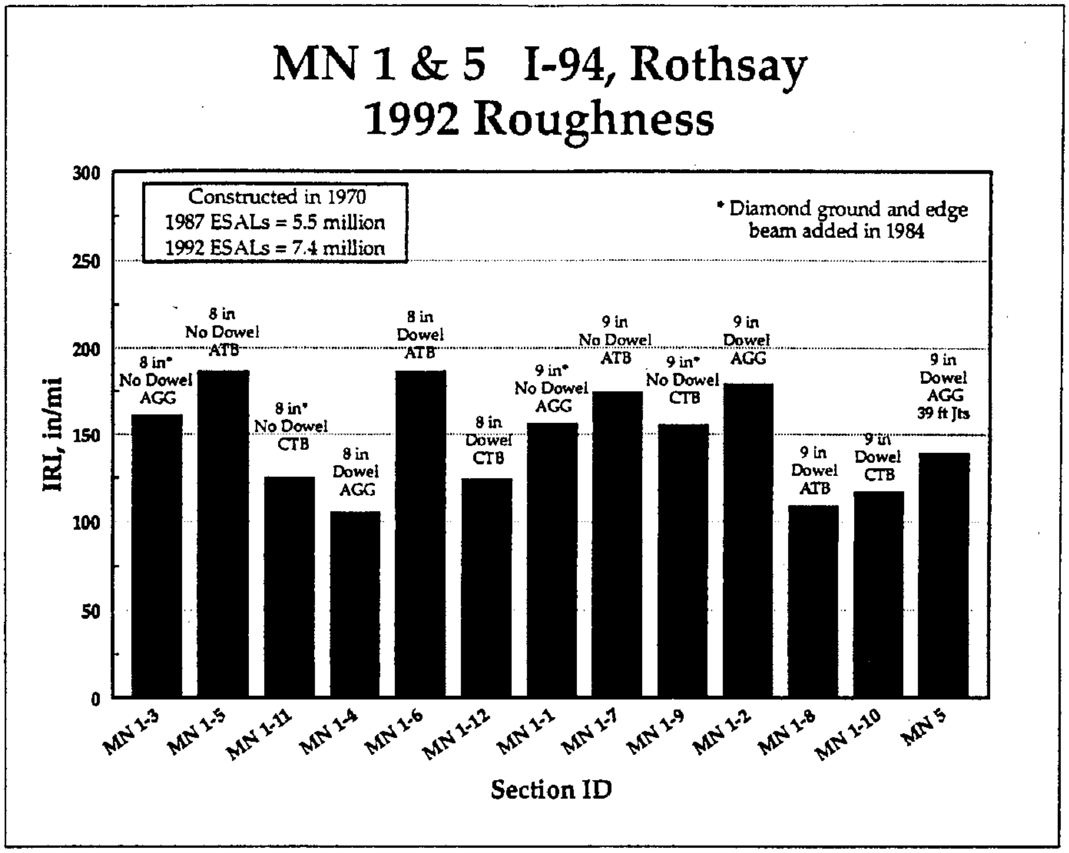
In addition, table 7 shows that longitudinal cracking slightly increased over the 5-year interval for the sections constructed on treated bases. Also, the amount of transverse joint spalling greatly increased for every section over the 5-year period. An examination of the 1987 and 1992 field data collection sheets indicates that this is primarily due to the deterioration of joints at the point where longitudinal cracks intersect. Some signs of D-cracking were also observed on these sections.

Summary

The performance exhibited by the sections in this project may be summarized as follows:

- The pavement sections placed on treated bases exhibit substantially more longitudinal cracking than the pavement sections constructed on aggregate bases. However, the levels of transverse cracking for each base type are about the same.

- The doweled pavement sections had less faulting than the nondoweled sections, although the small dowel bars (1 in [25 mm]) did allow significant levels of faulting.



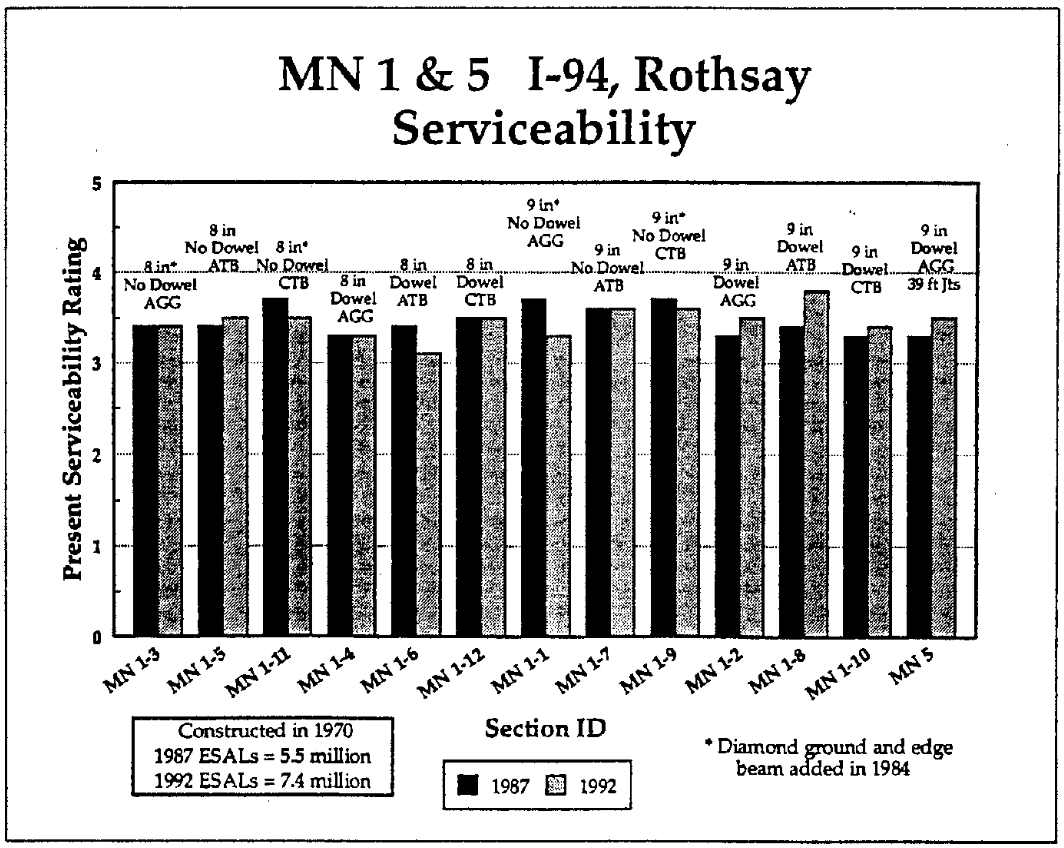


Figure 10. Roughness and serviceability for MN 1 & 5.

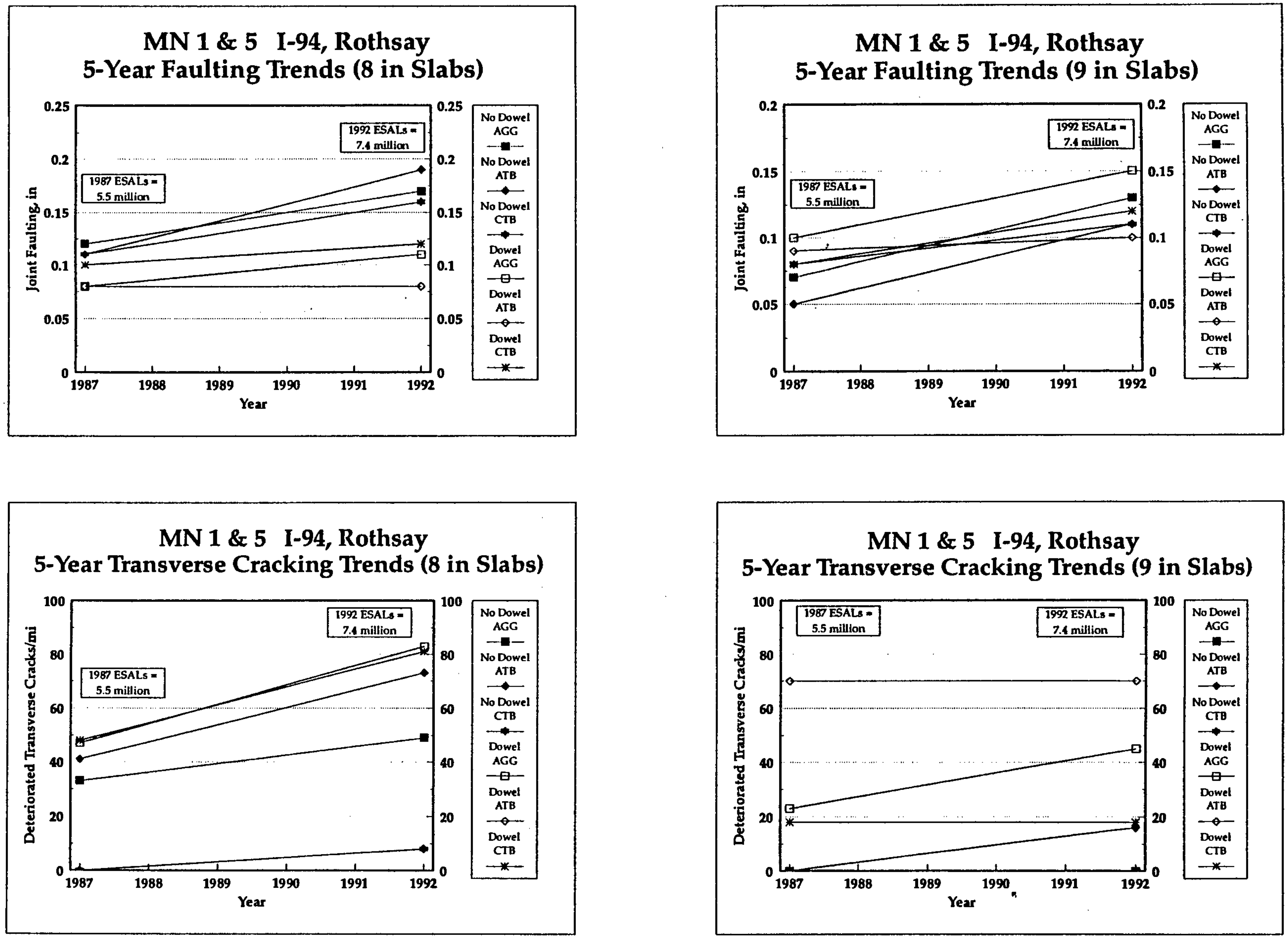


Figure 11. Faulting and transverse cracking 5-year performance trends for MN 1 & 5.

- The doweled sections often had more spalling and transverse cracking than the nondoweled sections, suggesting that the noncoated dowels had corroded and locked up many of the joints. The small amount of reinforcing steel (0.09 and 0.08 percent) may also be partially responsible for the deterioration of the transverse cracks.

- Increased slab thickness (from 8 in [203 mm] to 9 in [229 mm]) reduced faulting and cracking for the nondoweled sections but not for the doweled sections. Again, this result is probably related to the dowel bars locking up the joints and causing mid-slab cracks to open and deteriorate.

- The section with 27-ft (8.2-m) joint spacing displayed less transverse cracking and spalling than the longer 39-ft (12.2-m) pavement.

<u>Minnesota 2 (I-90, Albert Lea)</u>

A second experimental project in Minnesota is located on I-90, west of Albert Lea. The purpose of this experimental project is to evaluate the effect of tied concrete shoulders and widened traffic lanes on concrete pavement performance.[16,17] This project was included in the 1987 study and was re-evaluated in 1992.

The pavement sections were constructed in 1977. Design variables include pavement type (JRCP and JPCP), shoulder type, and slab thickness. All sections have a widened inner lane (15 ft [4.6 m] wide) and are constructed on an aggregate base course over an AASHTO A-6 subgrade. With the exception of the inner lanes of MN 2-1 and 2-2, the sections all contain 1-in (25-mm) diameter dowel bars. The experimental design matrix is shown in table 8.

Table 8. Experimental design matrix for MN 2.

<table>
<tr><th></th><th>8-in (203-mm) Slab</th><th colspan="2">9-in (229-mm) Slab</th></tr>
<tr><th></th><th>PCC
Shoulder</th><th>AC
Shoulder</th><th>PCC
Shoulder</th></tr>
<tr><td>JPCP
13-16-14-19-ft
(4.0-4.9-4.3-5.8-m)
Skewed Joints</td><td>MN 2-2</td><td></td><td>MN 2-1</td></tr>
<tr><td>JRCP (0.09% steel)
27-ft (8.2-m)
Skewed Joints</td><td></td><td>MN 2-3
MN 2-4</td><td></td></tr>
</table>

MN 2-1 and 2-3 have 6-in (152-mm) AGG base, MN 2-2 and 2-4 have 5-in (127-mm) AGG base.
All sections contain 15-ft (4.6-m) inside lane and no subbase.

Traffic Loadings

Through 1992, this pavement, a four-lane divided roadway, had sustained 4.2 million 18-kip (80 kN) ESAL applications in the outer lane, as compared to a total of 2.8 million ESAL applications in 1987. The two-way ADT for this roadway is 5,000 vehicles per day, which includes 22 percent heavy trucks.

Performance

A summary of the performance of the MN 2 sections is provided in table 9 and in figure 12. Because of the many confounding factors, it is difficult to make direct comparisons between the performance of these sections. A direct comparison of MN 2-1 (9-in [229-mm] slab) and MN 2-2 (8-in [203-mm] slab) is possible, but an examination of the data indicates that there is no appreciable difference in performance. The levels of joint faulting and transverse cracking are virtually the same, although some differences in spalling and longitudinal cracking are apparent. The longitudinal cracking on MN 2-1 is believed to be the result of the construction of the longitudinal joint.

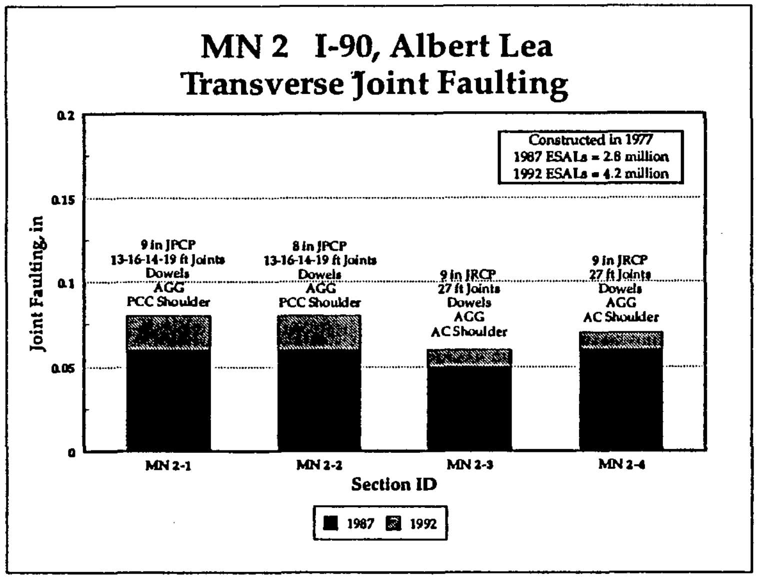
Table 9. Summary of 1987 and 1992 outer lane performance data for MN 2.

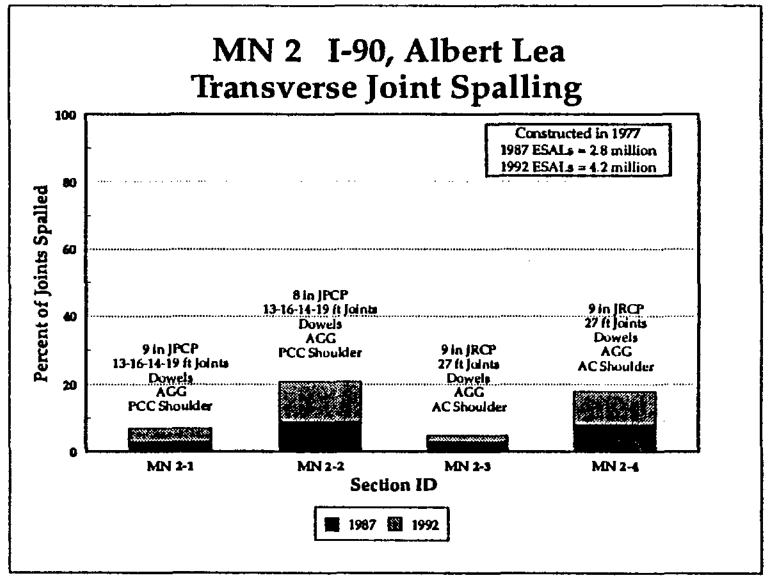
		8-in (203-mm) Slab		9-in (229-mm) Slab					
		PCC Shoulder		AC Shoulder				PCC Shoulder	
		1987 Survey	1992 Survey	1987 Survey		1992 Survey		1987 Survey	1992 Survey
Age at Survey, Years		10	15	10		15		10	15
ESAL's, millions		2.8	4.2	2.8		4.2		2.8	4.2
JPCP 13-16-14-19-ft (4.0-4.9-4.3-5.8-m) Skewed Joints		MN 2-2	MN 2-2					MN 2-1	MN 2-1
	Faulting, in	0.06	0.08					0.06	0.08
	Det. Tr. Crks/mi	0	5.0					0	0
	% Slabs with Tr. Crks	0	1					0	0
	Long. Cracks, ft/mi	150	150					746	932
	% Joints Spalled	9	21					3	7
	PSR	3.9	3.9					3.8	4.0
	IRI, in/mi	—	142					—	140
JRCP (0.09% steel) 27-ft (8.2-m) Skewed Joints				MN 2-3	MN 2-4	MN 2-3	MN 2-4		
	Faulting, in			0.05	0.06	0.06	0.07		
	Det. Tr. Crks/mi			0	5	10	5		
	Long. Cracks, ft/mi			0	0	142	0		
	% Joints Spalled			3	8	5	18		
	PSR			4.0	4.0	4.0	3.9		
	IRI, in/mi			—	—	113	153		

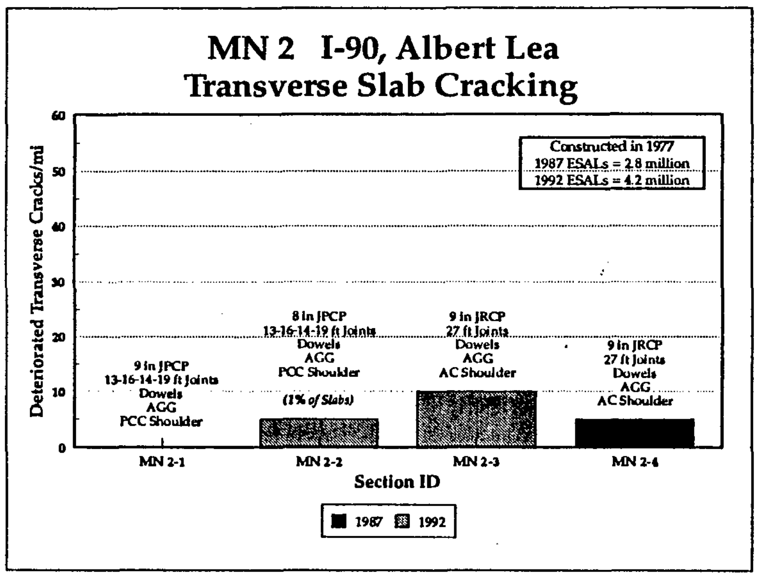
MN 2-1 and 2-3 have 6-in AGG base, MN 2-2 and 2-4 have 5-in AGG base.
All sections contain 15-ft inside lane and no subbase.
Every section contains 1-in dowels at transverse joints.

1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km

In assessing the performance of the different pavement types, the JRCP sections appear to be performing slightly better than the JPCP sections. The JRCP sections have slightly less faulting, less joint deterioration, and less longitudinal cracking.







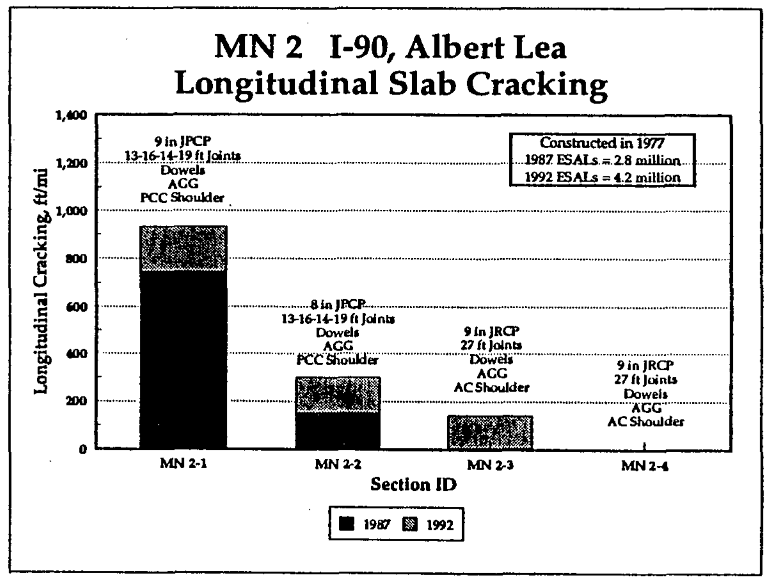


Figure 12. Performance summary for MN 2.

However, these differences are not appreciable and the overall rideability of both JRCP and JPCP sections is similar.

The effect of the PCC shoulder on pavement performance cannot be readily determined because of the confounding factors. Fewer deteriorated transverse cracks are found in the JPCP sections (which have tied PCC shoulders), suggesting that the tied PCC shoulders are providing additional lateral support, but joint spacing confounds this observation. In any case, the potential impact of tied PCC shoulders on performance, in terms of less cracking and faulting, will probably not be apparent for several more years.

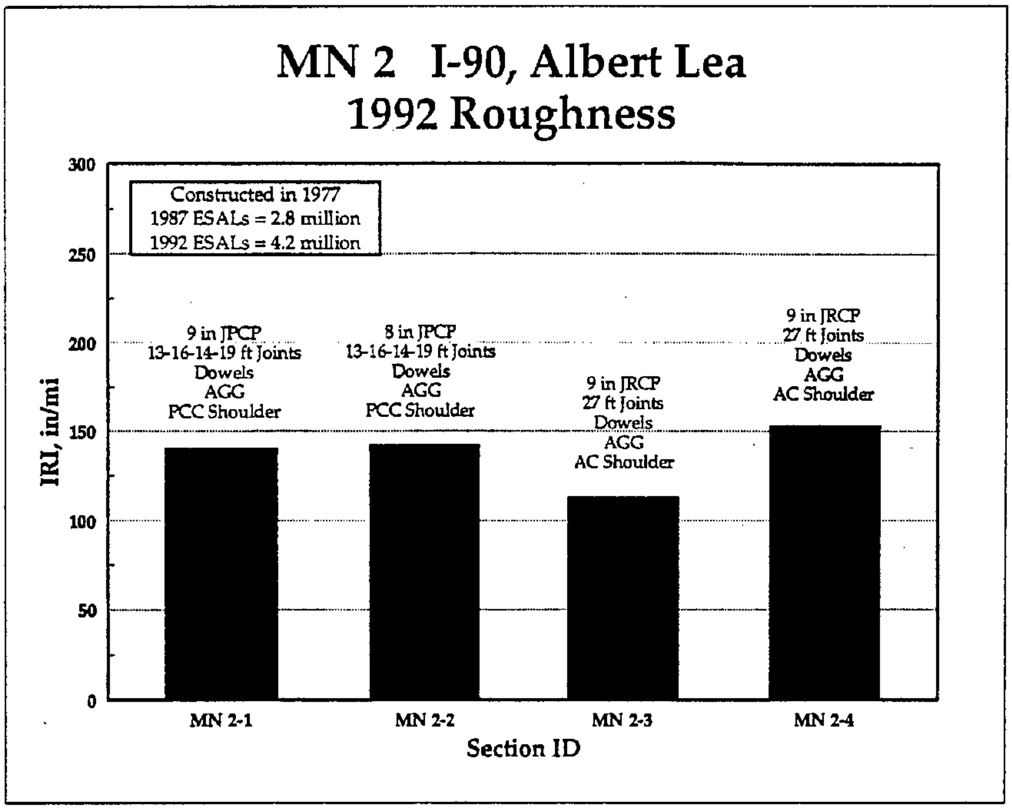
No conclusions can be drawn on the effect of the 15-ft (4.6-m) widened inside traffic lane, mainly because the evaluation concentrated on the outer traffic lane and no faulting measurements could be obtained on the inner lane. In addition, the researchers expect that the effects of a widened lane (especially on the inside lane where there is less traffic) will not be evident for many more years. However, the 3-ft (0.9-m) widening may be excessive in that it could contribute to longitudinal cracking; the Minnesota Department of Transportation currently suggests lane widenings of 1.5 to 2 ft (0.5 to 0.6 m).[17] Some states have experienced problems with longitudinal cracking on 16-ft (4.9-m) lanes, especially on thin pavement sections.

Roughness and Serviceability

Roughness and serviceability information for the MN 2 sections is presented graphically in figure 13. The IRI values are about the same for all sections, with MN 2-3 having slightly less roughness than the others. This is confirmed by the serviceability data, which shows MN 2-3 with a slightly higher serviceability value. Over the past 5 years, the serviceability of the MN 2 sections has remained virtually unchanged.

5-Year Performance Trends

As seen from figure 14, the MN 2 pavement sections are displaying a small increase in deterioration since being evaluated in 1987. Each of the sections shows a slight increase in joint faulting and joint spalling, with no single design exhibiting any obvious performance tendencies. In terms of transverse cracking, sections MN 2-2 (JPCP) and MN 2-3 (JRCP) have experienced a slight increase while the transverse cracking on sections MN 2-1 (JPCP) and MN 2-4 (JRCP) has remained the same. However, MN 2-1 and MN 2-3 show an increase in longitudinal cracking over the 5-year period, while MN 2-2 and MN 2-4 show no such increase. As mentioned before, the overall serviceability of these sections has changed very little since their evaluation in 1987.



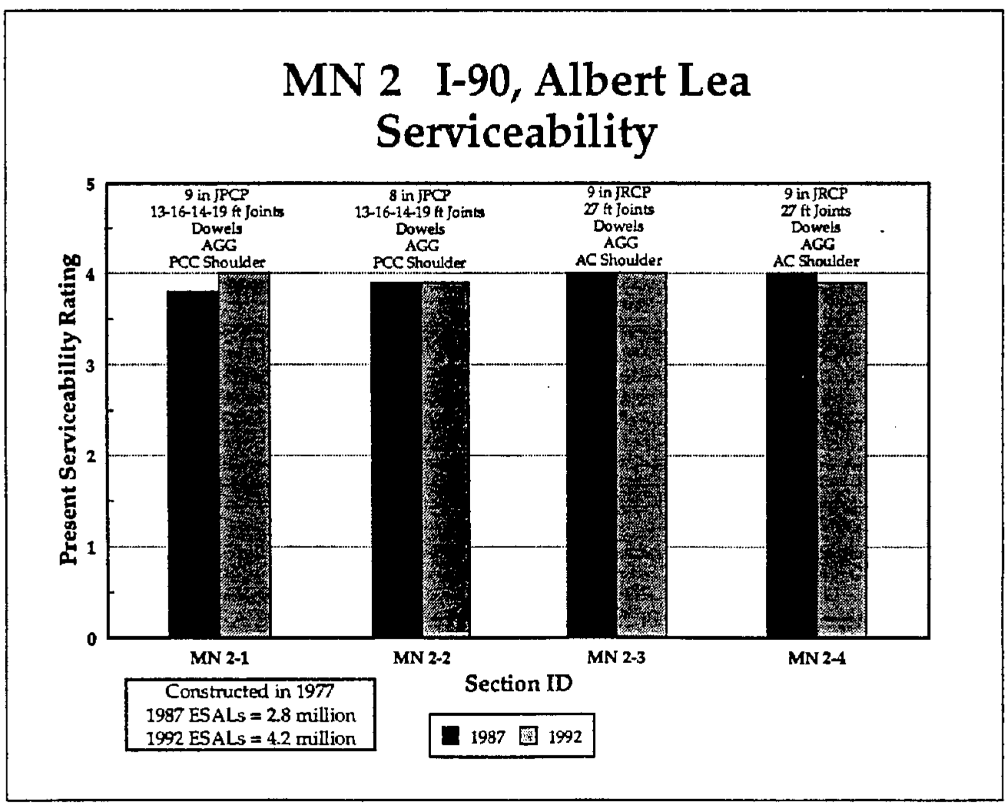


Figure 13. Roughness and serviceability for MN 2.

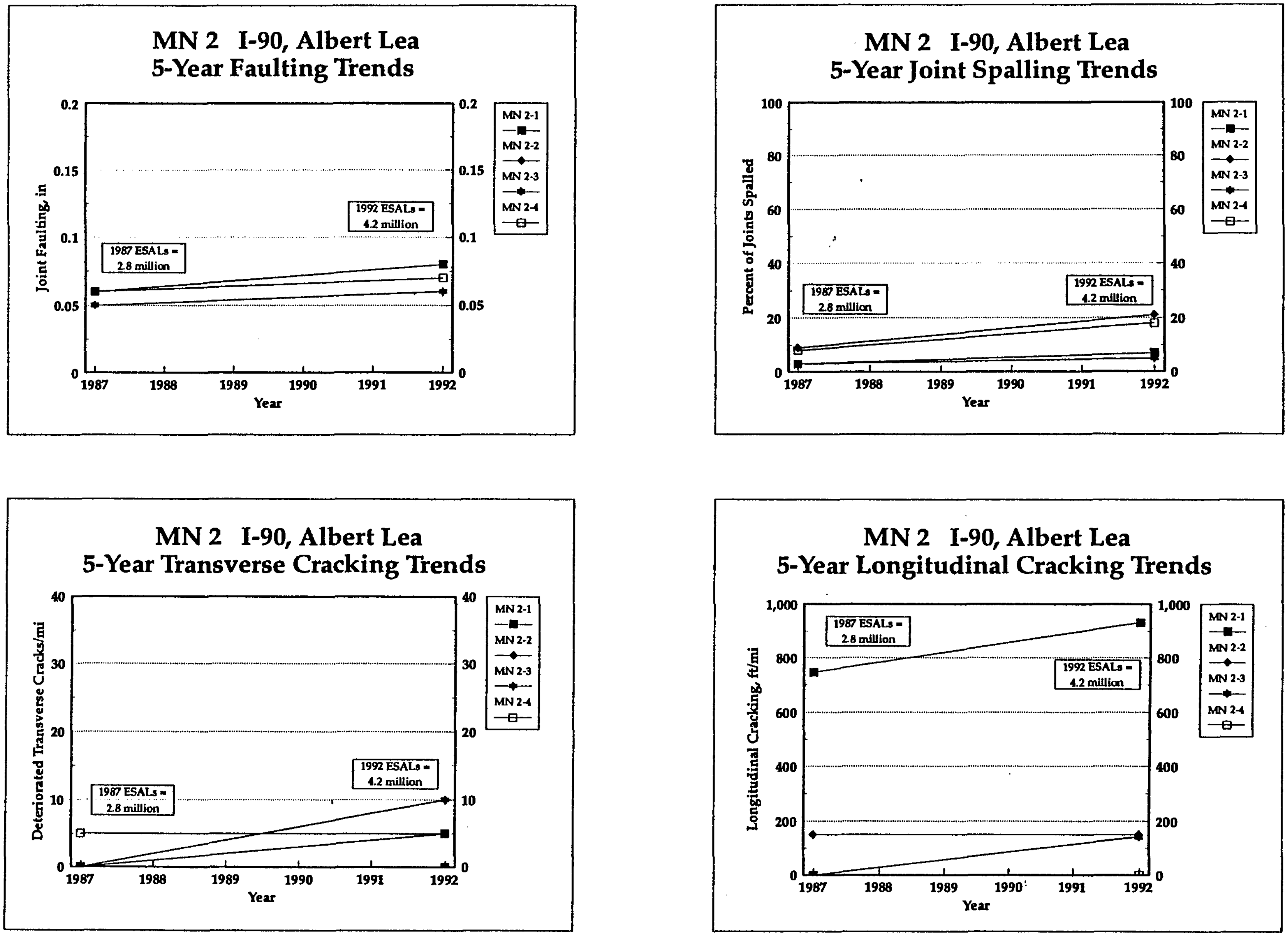


Figure 14. 5-year performance trends for MN 2.

Summary

The performance of these sections may be summarized as follows:

- MN 2-1, with a 9-in (229-mm) slab, shows less spalling and slightly less transverse cracking than MN 2-2, which consists of an 8-in (203-mm) slab. MN 2-1 does show more longitudinal cracking, but this is believed to be due to construction practices.

- The JRCP sections are exhibiting slightly better performance than the JPCP sections, although the differences are not considered significant.

- Because of the confounding factors, the effect of tied PCC shoulders and widened traffic lanes on concrete pavement performance could not be directly determined. The PCC shoulders may be providing additional lateral support to the mainline pavement, but their effect will not be known for several years.

- Slight increases in faulting, spalling, transverse cracking, and longitudinal cracking were observed over the last 5 years. At this time, no apparent trends exist that would suggest that one design is performing better than the others.

- The serviceability of each of these sections has remained virtually the same over the last 5 years.

Minnesota 3 (I-90, Austin)

This project, constructed in 1984 as part of I-90 in Austin, consists of a single pavement section. It is included in this study because it has a 14-ft (4.3 -m) widened outside lane and a 13.5-ft (4.1-m) widened inside lane. The pavement slab is a doweled, 9-in (229-mm) JRCP with 27-ft (8.2-m) skewed joints that rests on an aggregate base course and an AASHTO A-4 subgrade.

The design and performance of this section is summarized in table 10. The performance data indicate that, after 8 years of service, this pavement is still performing very well. The transverse joint faulting is virtually the same (actually a decrease of 0.01 in [0.25 mm], but this is considered within the range of acceptable variation), and no slab cracking or joint spalling exists. The roughness and serviceability values indicate a smooth-riding pavement.

Minnesota 4 (TH 15, New Ulm)

Minnesota 4 is a single pavement section located on Trunk Highway 15 near New Ulm. Built in 1986, this project is included in the study because of the 14-ft (4.3-m) widened outer lane. The pavement, located on a two-lane highway, is a 7.5-in (190-mm) JPCP constructed over an aggregate base course. The transverse joints are doweled, skewed, and spaced at 13-16-14-17-ft (4.0-4.9-4.3-5.2-m) intervals. The subgrade is an AASHTO A-2-6 material.

Table 10. Design and outer lane performance data for MN 3, MN 4, and MN 6.

	MN 3		MN 4		MN 6	
Pavement Type	JRCP 0.06% Steel		JPCP		JRCP 0.06% Steel	
Year Built	1984		1986		1983	
Thickness, in (mm)	9.0 (229)		7.5 (190)		8.0 (203)	
Joint Spacing, ft (m)	27 (8.2) Skewed		13-16-14-17 (4.0-4.9-4.3-5.2) Skewed		27 (8.2) Skewed	
Dowel Diameter, in (mm)	1.0 (25), epoxy-coated		1.0 (25), painted/greased		1.0 (25), painted/greased	
Base Type	4-in (102-mm) AGG		5-in (127-mm) AGG		4-in (152-mm) PATB (1.5-3.0% AC)	
Subbase Type	10-in (254-mm) AGG		None		4-in (102-mm) AGG	
Subgrade	AASHTO A-4		AASHTO A-2-6		AASHTO A-2-4	
Performance Data	1987	1992	1987	1992	1987	1992
Age at Survey, years	3	8	1	6	4	9
ESAL's, millions	1.5	3.7	0.2	0.9	0.8	2.0
Faulting, in	0.02	0.01	0.01	0.01	0.01	0.01
Det. Tr. Crks/mi	0	0	0	0	0	5
% Slabs with Tr. Crks	—	—	0	0	—	—
Long. Cracks, ft/mi	0	0	0	0	0	0
% Joints Spalled	0	0	0	1	0	3
PSR	3.8	4.2	4.8	4.4	4.5	4.2
IRI, in/mi	—	100	—	149	—	143

MN 3, 4, & 6 have a 14-ft widened outside lane.
MN 3 has a 13.5-ft widened inside lane.

1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km

Table 10 summarizes the design and performance of this section. This section is performing very well and is exhibiting no structural distress or deterioration. The serviceability has decreased since the 1987 survey, but the 1992 value still indicates a smooth-riding pavement surface.

Minnesota 6 (TH 15, Truman)

Minnesota 6, built in 1983, is a single pavement section located on Trunk Highway 15 near Truman. This project, a two-lane highway, is included because it contains a permeable asphalt-treated base (PATB) course and a 14-ft (4.3-m) widened outer lane. The pavement is a doweled, 8-in (203-mm) JRCP with 27-ft (8.2-m) skewed joints. The section is constructed on an AASHTO A-2-4 subgrade.

The design and performance of this section is summarized in table 10. This section is still in excellent condition, and is exhibiting minimal faulting and no longitudinal cracking. The only changes in performance from 1987 are the development of some transverse joint spalling and the deterioration of a few transverse cracks. The roughness and serviceability values are consistent with a smooth-riding pavement.

Minnesota 7 (TH 36, Roseville)

This experimental project, located on Trunk Highway 36 in Roseville, was constructed in 1958 with a range of design variables to determine the combination that provided the best performance.[18] Design variables for the study included pavement type (JRCP vs. JPCP), joint spacing (33 ft [10.1 m] vs. 65 ft [19.8 m] for JRCP and 15 ft [4.6 m] vs. 20 ft [6.1 m] for JPCP), load transfer (doweled vs. nondoweled, JPCP only), and base type (two types of aggregate bases). However, because the JRCP slabs were at some time sawed into shorter panel lengths, and because extensive joint repairs had been performed on the JRCP test sections, only the JPCP sections were evaluated under this study.

The design matrix for the sections originally selected for the study is shown in table 11. This table shows two levels of joint spacing (15 ft [4.6 m] vs. 20 ft [6.1 m]), load transfer (dowels vs. no dowels), and base type (Base A—3 in [76 mm] of gravel over 12 in [305 mm] of sand-gravel vs. Base B—3 in [76 mm] of gravel, 3 in [76 mm] of sand-gravel, and 3 in [76 mm] of gravel over 9 in [229 mm] of sand-gravel). However, upon conducting the surveys, the sections constructed on Base A (sections 7-1, 7-2, 7-5, and 7-6) could not be evaluated because nearly every transverse joint had been repaired.

Table 11. Experimental design matrix for selected sections on MN 7.

		9-in (229-mm) JPCP 15-ft (4.6-m) Joints	9-in (229-mm) JPCP 20-ft (6.1-m) Joints
AGG Base A	No Dowels	7-2*	7-1*
	1-in (25-mm) Dowels	7-5*	7-6*
AGG Base B	No Dowels	7-10 7-18	7-9 7-17
	1-in (25-mm) Dowels	7-15 7-23	7-16 7-24

* These sections were not surveyed because of extensive joint repairs.
Base A—3 in (76 mm) gravel and 12 in (305 mm) sand-gravel.
Base B—3 in (76 mm) gravel, 3 in (76 mm) sand-gravel, 3 in (76 mm) gravel, and 9 in (229 mm) sand-gravel.
All sections have an AASHTO A-4 subgrade.

For each doweled pavement section, three different types of 1-in (25-mm) diameter dowels were used: oiled, rust-proofed, and sleeved dowels. These dowels were placed at alternating joints within the section. Because of this, and the fact that each of these experimental sections is very short (225 to 300 ft [69 to 91 m]), it is difficult to fully evaluate the effect of the different types of dowel bars on pavement performance.

Traffic Loadings

This four-lane divided roadway is currently subjected to a two-way ADT of 48,000 vehicles per day, including 5 percent heavy trucks (FHWA classification 5 and greater). From its construction in 1958 and through 1992, this pavement has carried approximately 6.9 million ESAL applications in the outer traffic lane.

Performance

The performance data for the surveyed sections is presented in table 12 and displayed graphically in figure 15. Direct performance comparisons can be made between doweled and nondoweled sections and between the 15-ft (4.6-m) and the 20-ft (6.1-m) JPCP slabs.

Perhaps the most conspicuous observation from table 12 and figure 15 is that dowel bars are quite effective in reducing joint faulting. The nondoweled sections averaged 0.11 in (2.80 mm) of faulting, whereas the doweled sections averaged only 0.01 in (0.03 mm). The sections containing dowel bars did show slightly higher levels of joint spalling, but the difference is not considered significant.

Based on the available data, there does not appear to be any significant differential effect of these two joint spacings on pavement performance. The levels of faulting and spalling are approximately the same for both the 15-ft (4.6-m) and the 20-ft (6.1-m) slabs, although some of the 20-ft (6.1-m) sections did exhibit a small amount of transverse and longitudinal cracking. Nevertheless, there does not appear to be any significant differences in performance between the two designs.

Although an initial examination of the data in figure 15 appears to indicate that some directional effects in the performance of the sections may exist, a closer examination reveals some conflicting trends. For example, in terms of faulting, the westbound nondoweled sections are performing better than the eastbound nondoweled sections. However, in terms of spalling, the eastbound sections are performing better than the westbound sections. Yet there is no indication that these sections were rehabilitated at different times. No significant differences are noted by direction for transverse or longitudinal cracking.

Table 13 provides a summary of the performance of the various types of dowels that are used in the sections. These data are plotted in figure 16, which indicates that faulting is not significantly affected by the type of dowel, although joint spalling is

Table 12. Summary of 1992 outer lane performance data for MN 7.

			9-in (229-mm) JPCP 15-ft (4.6-m) Joints		9-in (229-mm) JPCP 20-ft (6.1-m) Joints		Average
			1992 Survey		1992 Survey		1992 Survey
Age at Survey, years			34	34	34	34	34
ESAL's, millions			6.9	6.9	6.9	6.9	6.9
AGG Base B	No Dowels		7-10	7-18	7-9	7-17	
		Faulting, in	0.07	0.16	0.07	0.15	0.11
		Det. Tr. Crks/mi	0	0	18	0	4
		% Slabs with Tr. Crks	0	0	6	0	1
		Long. Cracks, ft/mi	0	0	0	0	0
		% Joints Spalled	50	31	39	31	38
		PSR	3.3	3.5	3.4	2.9	3.2
		IRI, in/mi	162	158	169	194	171
	1-in (25-mm) Dowels		7-15	7-23	7-16	7-24	
		Faulting, in	0.01	0.01	0.01	0.01	0.01
		Det. Tr. Crks/mi	0	0	0	18	4
		% Slabs with Tr. Crks	0	0	0	6	2
		Long. Cracks, ft/mi	0	0	176	0	44
		% Joints Spalled	59	31	75	39	51
		PSR	3.1	3.3	3.0	3.2	3.2
		IRI, in/mi	193	183	201	179	189
Average		Faulting, in	0.04	0.08	0.04	0.08	
		Det. Tr. Crks/mi	0	0	9	9	
		% Slabs with Tr. Crks	0	0	3	3	
		Long. Cracks, ft/mi	0	0	0	0	
		% Joints Spalled	54	31	57	35	
		PSR	3.2	3.4	3.2	3.0	
		IRI, in/mi	178	170	185	186	

Base B—3-in (76-mm) gravel, 3-in (76-mm) sand-gravel, 3-in (76-mm) gravel, and 9-in (229-mm) sand-gravel.
All sections have an AASHTO A-4 subgrade.

1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km

Table 13. Summary of dowel performance on MN 7.

Dowel Type	Faulting, in (mm)			% Joints Spalled		
	15-ft (4.6-m) Joints	20-ft (6.1-m) Joints	Average	15-ft (4.6-m) Joints	20-ft (6.1-m) Joints	Average
Oiled	0.012 (0.30)	0.006 (0.15)	0.009 (0.23)	9	36	23
Rust-Proofed	0.012 (0.30)	0.008 (0.20)	0.011 (0.28)	64	55	59
Sleeved	0.025 (0.64)	0.022 (0.56)	0.024 (0.61)	70	90	80

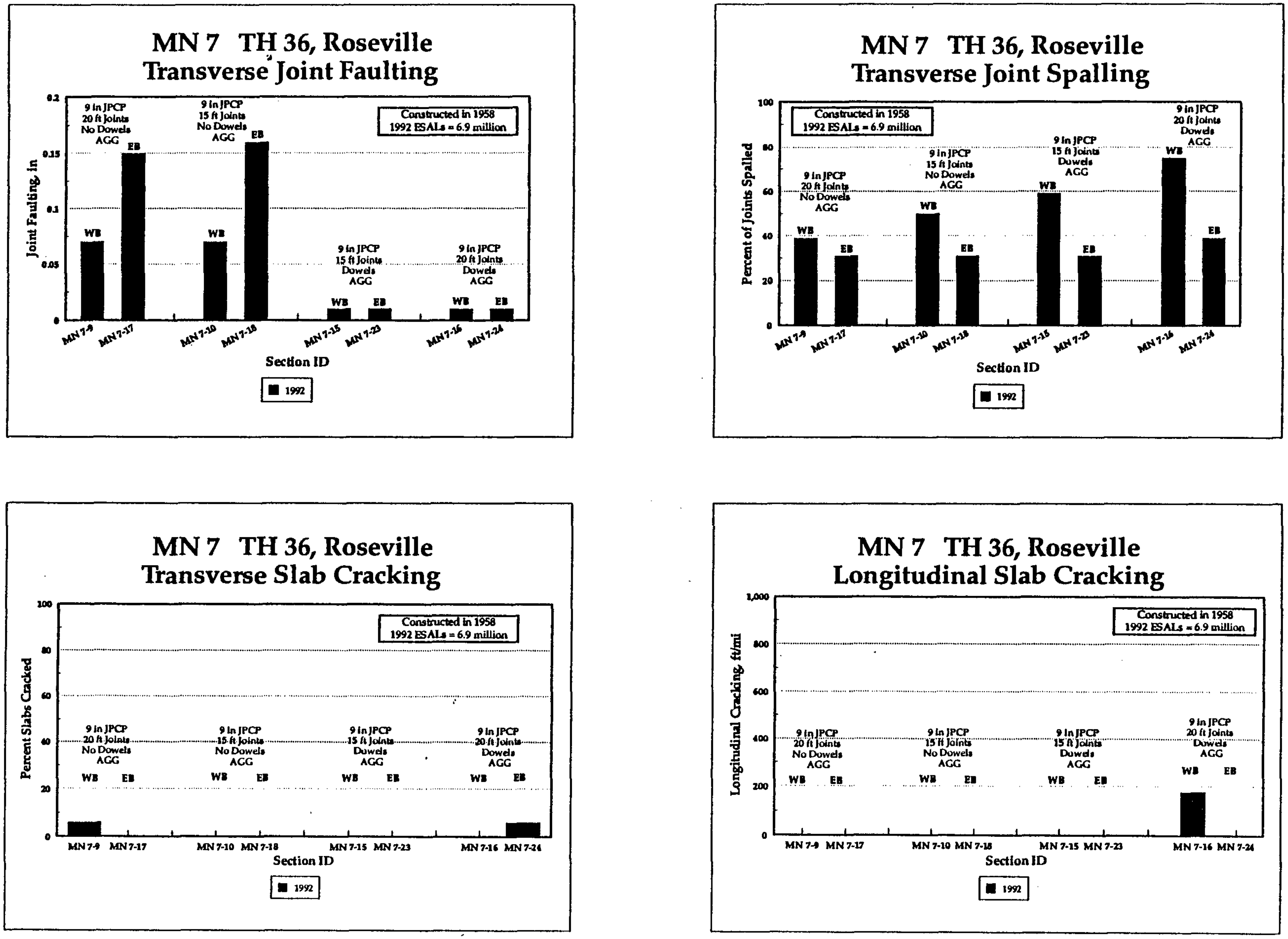


Figure 15. Performance summary for MN 7.

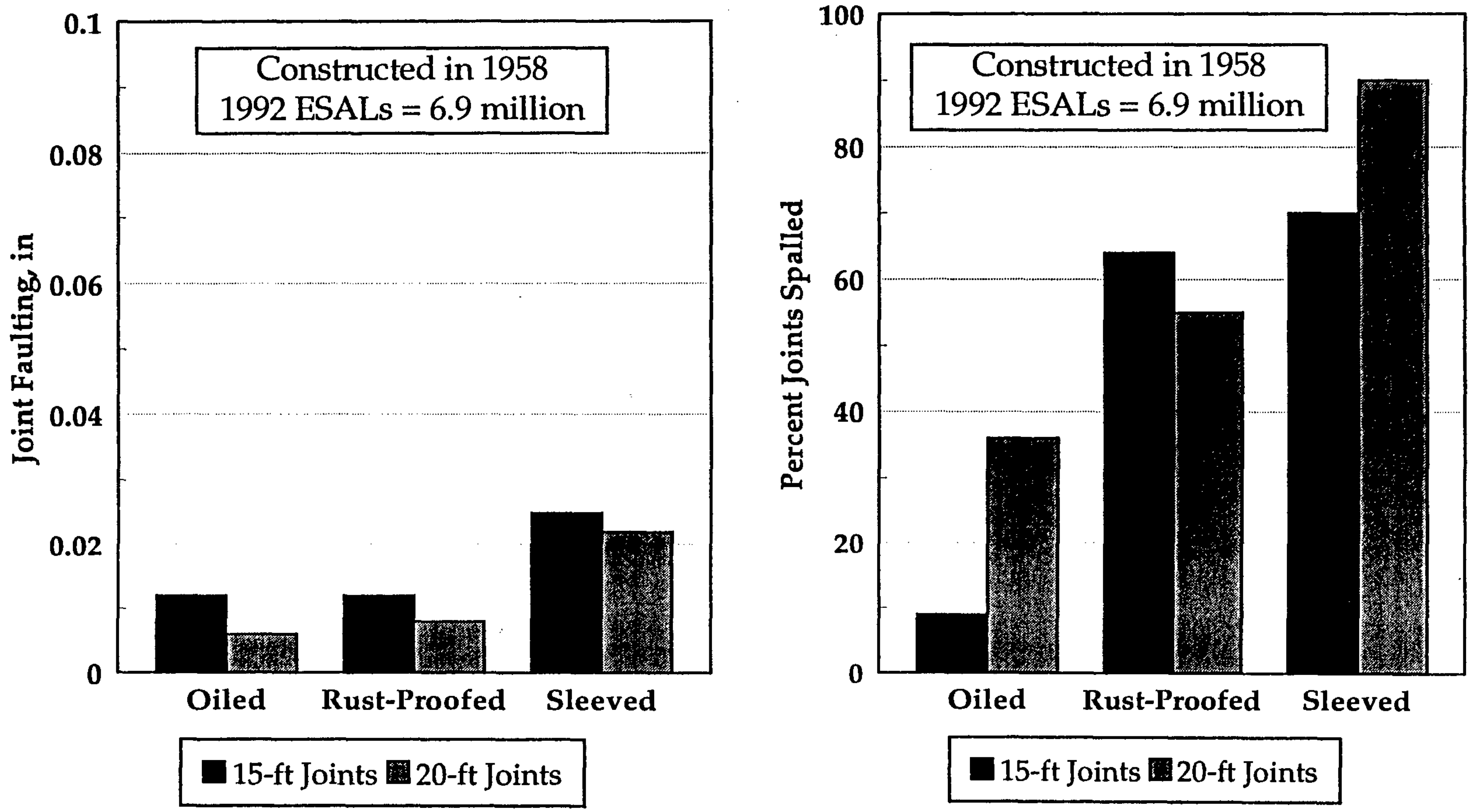


Figure 16. Performance of dowel coatings for MN 7.

affected. The joints with oiled dowels exhibited much less medium- and high-severity joint spalling than those joints with rust-proofed or sleeved dowels. The joints containing the sleeved dowels were badly deteriorated, and several had received full- and partial-depth repairs. It should be recalled that the different dowel types were placed at alternating joints.

Research conducted by the Minnesota Department of Transportation indicated that the sleeved dowels performed the worst.[18] That research also indicated that the 15- and 20-ft (4.6- and 6.1-m) doweled JPCP sections performed better than not only the nondoweled JPCP, but also better than the JRCP designs.[18] As previously mentioned, the long-jointed JRCP slabs had received extensive patching at nearly every transverse joint.

Roughness and Serviceability

Figure 17 presents the roughness and serviceability data for the MN 7 sections. This figure shows that the pavement sections are at or approaching critical roughness and serviceability levels. The presence of dowels or of different joint spacings does not appear to have any effect on the overall pavement roughness levels or serviceabilities.

Summary

A summary of the performance of the eight JPCP sections on T.H. 36 near Roseville is given below:

- Dowel bars were definitely effective in reducing transverse joint faulting. Joint faulting averaged 0.11 in (2.8 mm) for the nondoweled joints and only 0.01 in (0.25 mm) for the doweled joints.

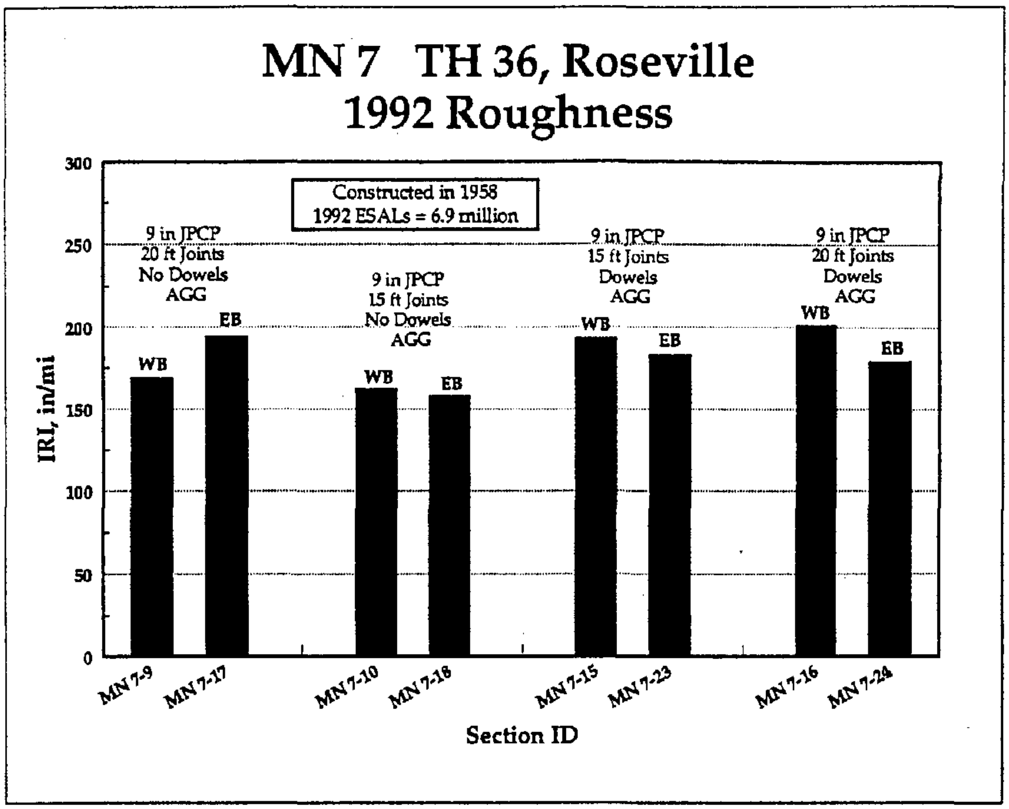
- No appreciable difference in performance between the 15-ft (4.6-m) sections and the 20-ft (6.1-m) sections was observed.

- Faulting of the nondoweled sections was more severe in the eastbound lanes than in the westbound lanes. However, joint spalling was more severe in the westbound lanes.

- An evaluation of the various types of dowel coating indicates that the joints containing sleeved dowels exhibited substantial joint spalling. The joints containing the oiled dowels displayed the least amount of spalling. No significant differences in joint faulting of the different dowel coatings was determined.

Performance Summary of Projects in Dry-Nonfreeze Zone

Two States located in the dry-nonfreeze environmental region contribute projects to the study: Arizona and California. One experimental project is located in Arizona and two are located in California. In addition, one single section project from Arizona and five single section projects from California are included.



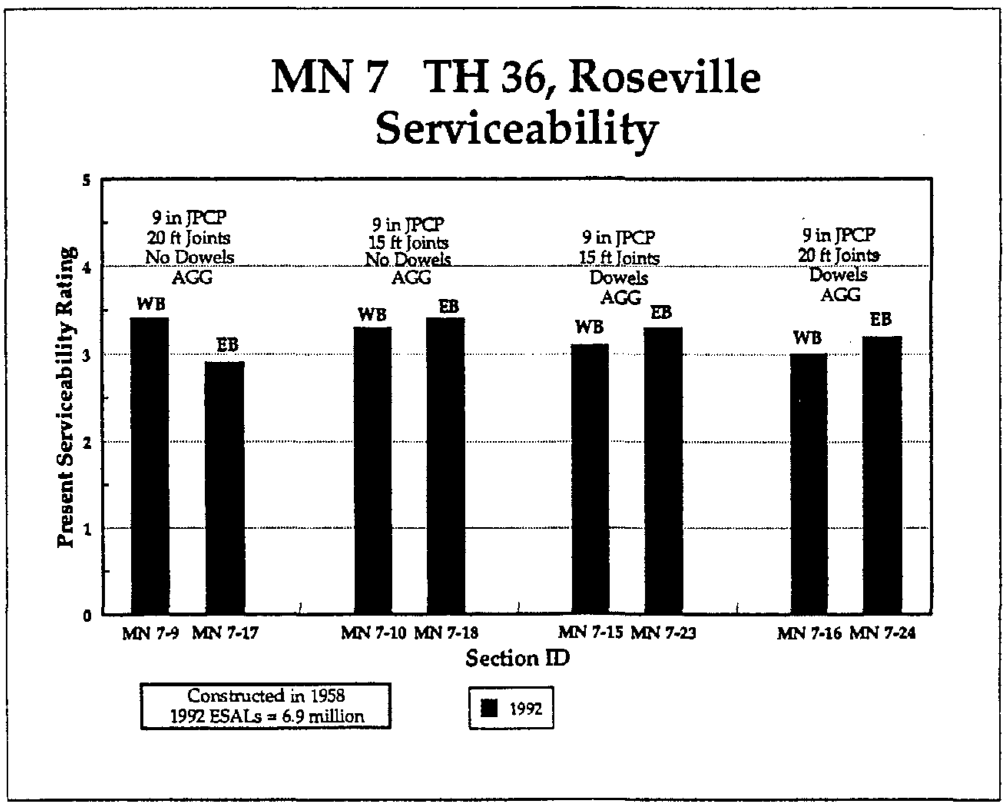


Figure 17. Roughness and serviceability for MN 7.

Climatic indicators for this region include a Corps of Engineers Freezing Index of 0, a Thornthwaite Moisture Index range of -10 to -30, and annual precipitation ranging from 8 to 17 in (203 to 432 mm). The highest average monthly maximum temperature ranges from 89 °F (32 °C) to 105 °F (41 °C), while the lowest average monthly minimum temperature ranges from 36 °F (2 °C) to 41 °F (5 °C).

Arizona 1 (S.R. 360, Phoenix)

A series of experimental concrete pavement sections were constructed on State Route 360 in Phoenix, Arizona, over a number of years during the 1970's and early 1980's.[19] Experimental design features include base type, slab thickness, shoulder type, and drainage. All of the sections are nondoweled JPCP with random, skewed transverse joints spaced at 13-15-17-15-ft (4.0-4.6-5.2-4.6-m) intervals. The subgrade varies from an AASHTO A-4 to an A-6. The different design sections are summarized in table 14. Since each of these sections was constructed in different years, they each have accumulated different ESAL applications.

Table 14. Experimental design matrix for AZ 1.

		No Edge Drains		Edge Drains
		AC Shoulder	PCC Shoulder	PCC Shoulder
9-in (229-mm) JPCP	6-in (152-mm) CTB (4.3% cement)	AZ 1-1 (1972)		
	4-in (102-mm) LCB (6.9% cement)		AZ 1-6 (1981)	AZ 1-7 (1981)
11-in (279-mm) JPCP	No Base		AZ 1-5 (1979)	
13-in (330-mm) JPCP	No Base		AZ 1-2 (1975) AZ 1-4 (1979)	

AZ 1-1 has a 4-in (102-mm) aggregate subbase on an A-4 subgrade.
All other sections have no subbase and an A-6 subgrade.

Traffic Loadings

The traffic for these sections varies considerably over the length of the project. The roadway, which was originally constructed as a four-lane divided highway, now has a third traffic lane added in several places. Being in an urban environment, there are numerous major interchanges. The traffic volumes and the 18-kip (80-kN) ESAL applications for each section are given in table 15.

Performance

Table 16 summarizes the performance data for the AZ 1 sections. Figure 18 presents those data in graphical format. Overall, these sections are performing very well, with only slight differences in performance. However, each section is a

Table 15. Traffic summary for AZ 1 sections.

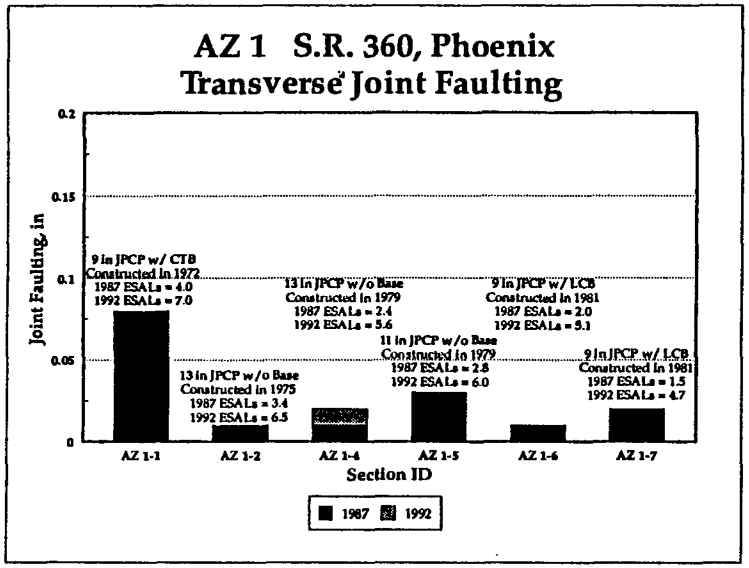
Section	Year Built	1992 2-way ADT	1992 % Trucks	ESAL's, millions (outer lane) 1987	ESAL's, millions (outer lane) 1992
AZ 1-1	1972	119,000	3.5	4.0	7.0
AZ 1-2	1975	127,000	3.5	3.4	6.5
AZ 1-4	1979	129,000	3.5	2.4	5.6
AZ 1-5	1979	129,000	3.5	2.8	6.0
AZ 1-6	1981	104,000	3.5	2.0	5.1
AZ 1-7	1981	104,000	3.5	1.5	4.7

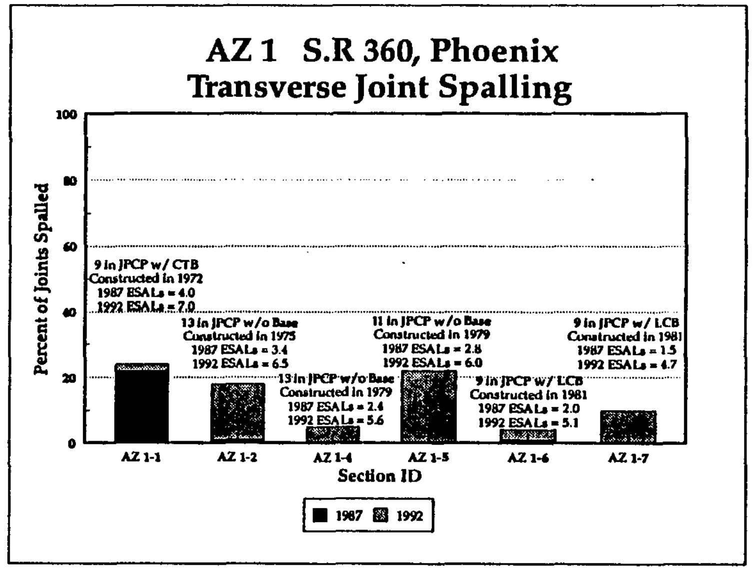
Table 16. Summary of 1987 and 1992 outer lane performance data for AZ 1.

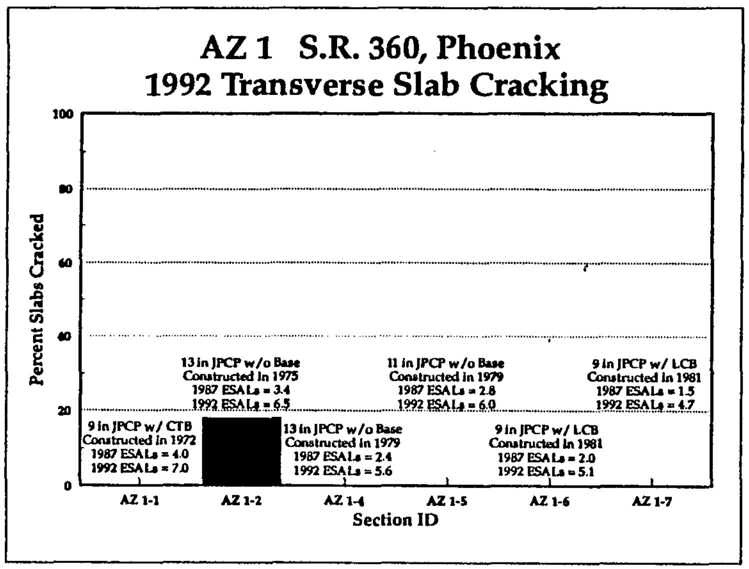
			No Edge Drains				Edge Drains	
			AC Shoulder		PCC Shoulder		PCC Shoulder	
			1987	1992	1987	1992	1987	1992
9-in (229-mm) JPCP	6-in (152-mm) CTB (4.2%)		AZ 1-1	AZ 1-1				
		Age at Survey, years	15	20				
		ESAL's, millions	4.0	7.0				
		Faulting, in	0.08	0.08				
		Det. Tr. Crks/mi	0	0				
		% Slabs Cracked	0	0				
		Long. Cracks, ft/mi	233	278				
		% Joints Spalled	22	24				
		PSR	3.4	3.9				
		IRI, in/mi	—	105				
	4-in (102-mm) LCB (6.9 %)				AZ 1-6	AZ 1-6	AZ 1-7	AZ 1-7
		Age at Survey, years			6	11	6	11
		ESAL's, millions			2.0	5.1	1.5	4.7
		Faulting, in			0.01	0.01	0.02	0.02
		Det. Tr. Crks/mi			0	0	0	0
		% Slabs Cracked			0	0	0	0
		Long. Cracks, ft/mi			0	0	0	0
		% Joints Spalled			1	4	0	10
		PSR			3.5	3.5	3.8	3.6
		IRI, in/mi			—	123	—	135
11-in (270-mm) JPCP	No Base				AZ 1-5	AZ 1-5		
		Age at Survey, years			8	13		
		ESAL's, millions			2.8	6.0		
		Faulting, in			0.03	0.03		
		Det. Tr. Crks/mi			0	0		
		% Slabs Cracked			0	0		
		Long. Cracks, ft/mi			0	0		
		% Joints Spalled			0	22		
		PSR			3.8	3.9		
		IRI, in/mi			—	102		
13-in (330-mm) JPCP	No Base				AZ 1-2 / AZ 1-4	AZ 1-2 / AZ 1-4		
		Age at Survey, years			12 / 8	17 / 13		
		ESAL's, millions			3.4 / 2.4	6.5 / 5.6		
		Faulting, in			0.01 / 0.01	0.01 / 0.02		
		Det. Tr. Crks/mi			0 / 0	69 / 0		
		% Slabs Cracked			0 / 0	18 / 0		
		Long. Cracks, ft/mi			0 / 0	0 / 20		
		% Joints Spalled			1 / 0	18 / 5		
		PSR			3.8 / 3.6	4.2 / 3.8		
		IRI, in/mi			— / —	111 / 122		

AZ 1-1 has a 4-in (102-mm) aggregate subbase on an A-4 subgrade.
All other sections have no subbase and an A-6 subgrade.

1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km







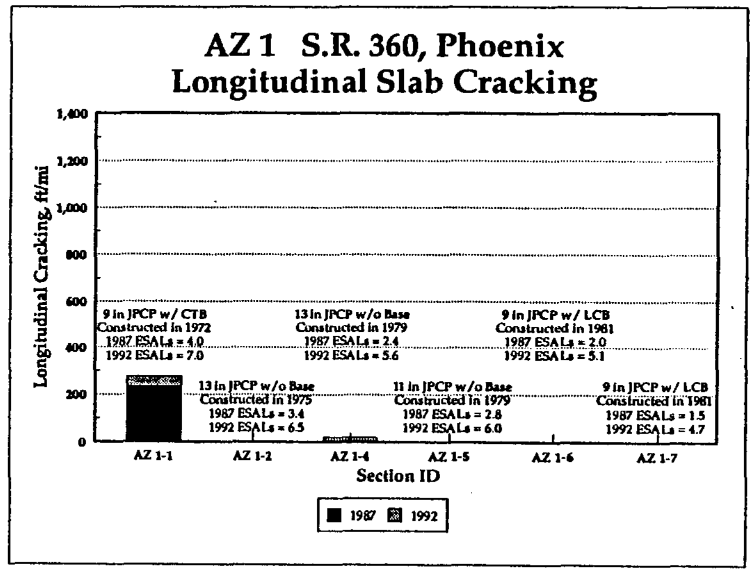


Figure 18. Performance summary for AZ 1.

different age and has been subjected to different traffic loadings, which makes direct comparisons difficult.

The effect of edge drains on performance can be directly compared by examining the performance of AZ 1-6 and 1-7. From these two sections, no observable benefit is derived from providing edge drains on these pavements in the Phoenix climate, although it is not known whether or not the edge drains that were constructed are still functioning properly.

A direct comparison of the effect of base type is not possible for these sections, as it is compounded by slab thickness, shoulder type, and subdrainage. The 9-in (229-mm) slab on CTB is observed to be performing the poorest of any of the sections in terms of faulting (0.08 in [2 mm]), spalling, and longitudinal cracking. However, this section is the oldest section in the project and has been subjected to the most ESAL applications.

Considering the combined effect of slab thickness and base type, however, does suggest some performance differences. The thicker slabs with no base performed about as well as the 9-in (229-mm) slabs with the LCB. Their faulting is comparable and their rideability appear to be very similar also.

An examination of the effect of slab thickness on performance is possible through looking at AZ 1-5 and comparing it to AZ 1-2 and 1-4. Overall, the performance of the different sections is similar, and there is little to differentiate between these sections, either in 1987 or in 1992. Section AZ 1-2, however, does exhibit some low-severity transverse cracking.

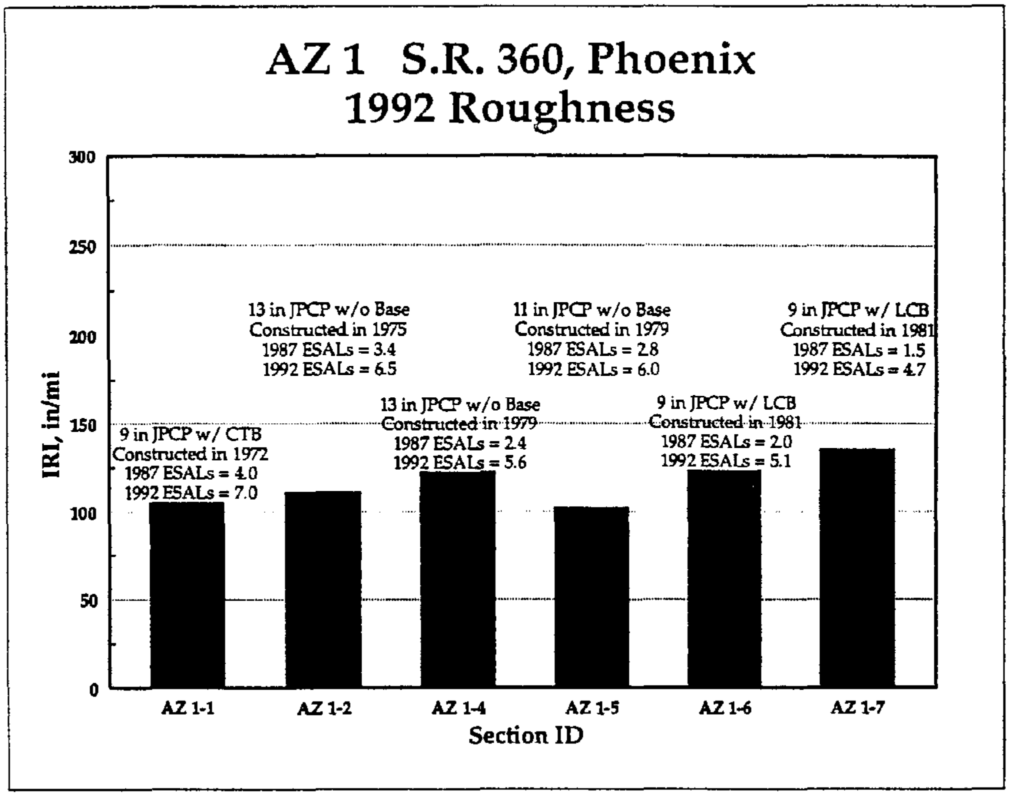
Roughness and Serviceability

Figure 19 shows the roughness levels and serviceability ratings for the AZ 1 sections. Overall, each section appears to be providing a smooth-riding pavement surface. The serviceability ratings indicate that the rideability has remained virtually the same since 1987. Surprisingly, the roughest sections in the project are the newest (AZ 1-6 and AZ 1-7).

5-Year Performance Trends

As shown in figure 20, none of the sections showed an appreciable increase in pavement distress over the period from 1987 to 1992. Faulting levels remained virtually the same over the 5-year period, and only one section (AZ 1-2) developed transverse cracking. Two sections (AZ 1-1 and AZ 1-4) exhibit longitudinal cracking, but the amount is not considered significant.

One performance indicator that has shown an increase over the 5-year period is transverse joint spalling. Every section shows some increase in spalling, although the increase is often not that substantial. Sections AZ 1-1 (24 percent of the joints) and AZ 1-2 (18 percent of the joints) display the most spalling, and these are also the oldest sections in the project. During the 1992 pavement surveys, the joint sealant was observed to be in poor condition for all sections.



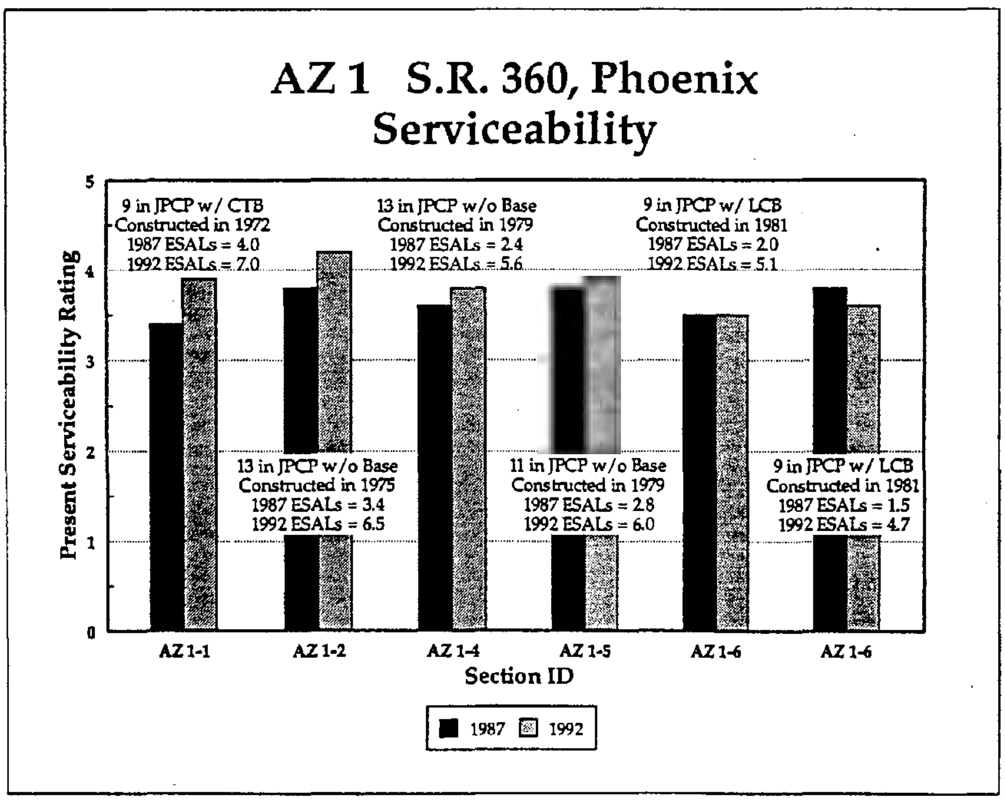


Figure 19. Roughness and serviceability for AZ 1.

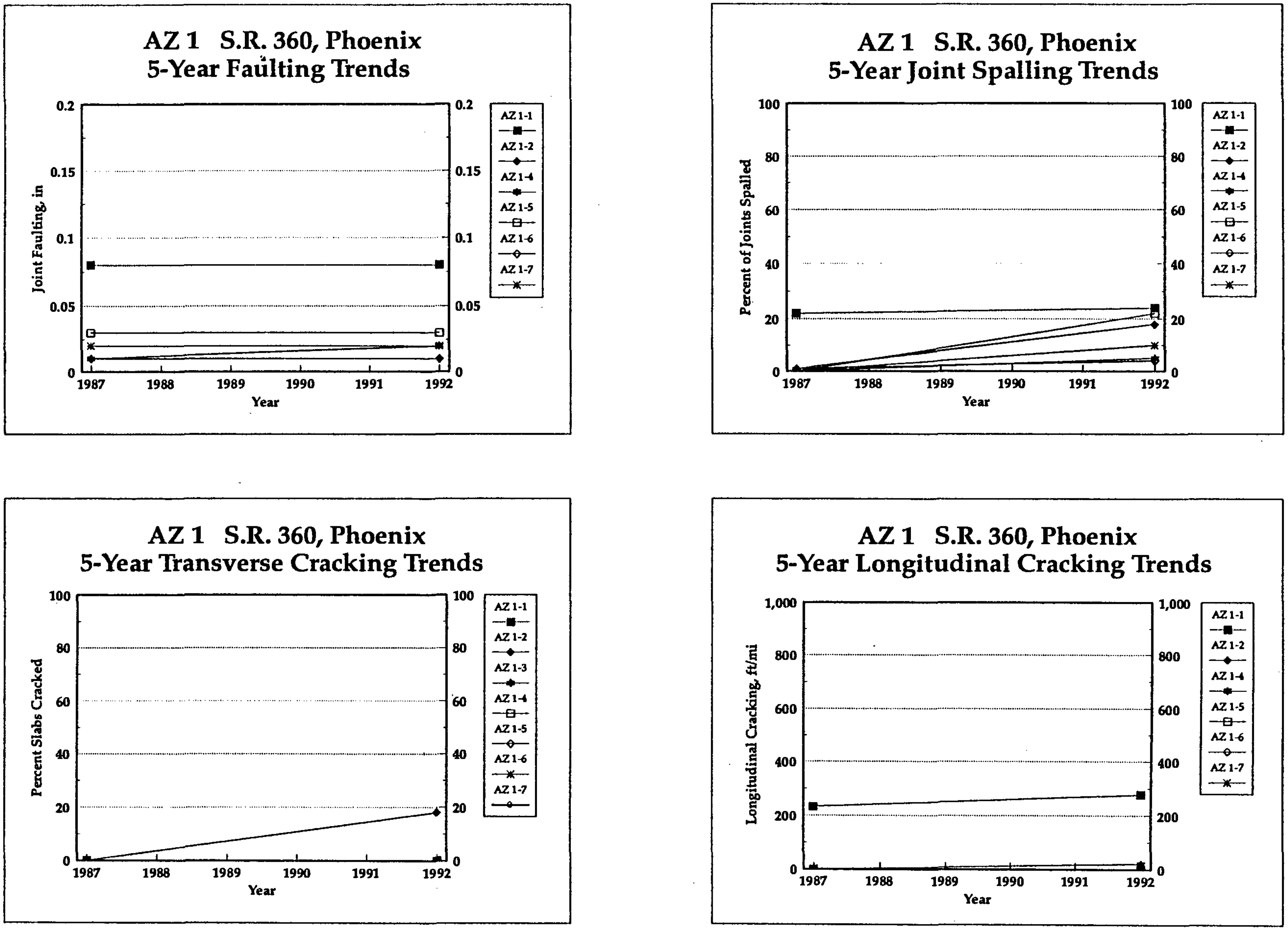


Figure 20. 5-year performance trends for AZ 1.

Summary

The performance of the AZ 1 sections is summarized below:

- For the dry climate of Arizona, no apparent advantages in performance are provided by the inclusion of edge drains.

- The effect of base type on pavement performance could not be directly determined. The 9-in (229-mm) section constructed on CTB is exhibiting the worst performance of all of the sections, but it is also the oldest.

- The 11- and 13-in (270- and 330-mm) sections placed directly on grade are performing about the same, and comparable to the 9-in (229-mm) sections placed on a base.

- The sections are exhibiting smooth-riding characteristics, as exhibited by the roughness and the serviceability ratings. Virtually no changes in the rideability values have occurred over the 5-year period.

- Transverse joint spalling is the only distress to significantly increase on the AZ 1 sections over the 5-year period. Much of the joint sealant material was observed to have hardened and deteriorated.

Arizona 2 (I-10, Phoenix)

This single pavement section is located on I-10 in Phoenix. Built in 1983, this section is included in the study because it represents one of the few JPCP sections in a dry climate that incorporates dowel bars (1.25 in [32 mm] epoxy-coated) in its design. The pavement consists of a 10-in (254-mm) slab over a 5-in (127-mm) lean concrete base (LCB) course containing 6.9 percent cement. Transverse joints are skewed and spaced at intervals of 13-15-17-15 ft (4.0-4.6-5.2-4.6-m). The subgrade is an AASHTO A-6.

The design and performance of this section is summarized in table 17. Most notably, a significant increase in transverse joint spalling has occurred from 1987 to 1992, as well as a small increase in longitudinal cracking. However, no transverse cracking has developed and joint faulting has not increased over the 5-year period. The serviceability rating has not changed since the 1987 survey and the current roughness level indicates a smooth-riding pavement surface.

California 1 (I-5, Tracy)

A set of experimental sections was constructed on I-5 near Tracy, California in 1971. Four different pavement designs were included to study the effect of slab thickness, joint spacing, and base type on pavement performance.[20-22] In addition, one section was constructed with high-strength concrete. All of the CA 1 sections are nondoweled JPCP designs with random, skewed, and nonsealed transverse joints. The subgrade soils range from an AASHTO A-1-a to an A-2-4. The experimental matrix for this project is shown in table 18.

Table 17. Summary of 1987 and 1992 outer lane performance data for AZ 2.

	AZ 2	
Pavement Type	JPCP	
Year Built	1983	
Thickness, in	10 (254 mm)	
Joint Spacing, ft	13-15-17-15 (4.0-4.6-5.2-4.6 m)	
Dowel Diameter, in	1.25 (32 mm)	
Base Type	5 in (127 mm) LCB (6.9% cement)	
Subbase	None	
Subgrade	AASHTO A-6	
Performance Data	1987 Survey	1992 Survey
Age at Survey, years	4	9
ESAL's, millions	2.9	9.6
Faulting, in	0.01	0.01
Deteriorated Tr. Crks, ft/mi	0	0
Long. Cracks, ft/mi	0	20
% Joints Spalled	0	39
PSR	3.6	3.6
IRI, in/mi	—	96

1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km

Table 18. Experimental design matrix for CA 1.

		12-13-19-18-ft Jts (3.7-4.0-5.8-5.5-m)		5-8-11-7-ft Jts (1.5-2.4-3.4-2.1-m)	8.4-in (213-mm) CRCP		
		8.4-in (213-mm) JPCP	11.4-in (290-mm) JPCP	8.4-in (213-mm) JPCP	Long. Bars	Long + Tr. Bars	Welded Wire Fabric
Normal Strength Concrete (5.5 bag)	CTB (4%)	CA 1-3* CA 1-4	CA 1-5* CA 1-6	CA 1-1* CA 1-2	CA 1-11 CA 1-12	CA 1-13 CA 1-14	CA 1-15 CA 1-16
	LCB (4 bag)	CA 1-7* CA 1-8					
High Strength Concrete (7.5 bag)	CTB (4%)	CA 1-9* CA 1-10					

* 1987 section.
All sections have a 5.4-in (137-mm) base and a 24-in (610-mm) aggregate subbase.
CA 1-5 = SHRP 67456, CA 1-14 = SHRP 67455

The odd-numbered sections shown in table 18 are all located in the northbound direction. Although not surveyed in 1987, replicate (even-numbered) sections in the southbound direction were also surveyed in 1992 as part of this study. In addition, several CRCP sections were included to evaluate the performance of different concrete pavement types.

Traffic Loadings

Through 1992, this pavement, a four-lane divided roadway, has sustained 11.9 million 18-kip (80 kN) ESAL applications in the outer lane, compared to a total of 7.6 million ESAL applications in 1987. The two-way ADT for this roadway is 13,000 vehicles per day, which includes 24 percent heavy trucks. While opened to traffic in 1971, these sections did not receive appreciable truck traffic loadings until 1976.

Performance

The performance of the CA 1 sections is summarized in table 19 and in figure 21. The influence of slab thickness on performance can be examined through a comparison of section CA 1-3 with section 1-5. The faulting and joint spalling exhibited by both of these sections is rather high, and there isn't any appreciable difference between them. Similar levels of faulting and spalling are also exhibited by the replicate sections (CA 1-4 and CA 1-6). However, the thickened slab section does display less cracking than the thinner section, which suggests that the thicker slab is effective at reducing slab cracking but not joint faulting.

The influence of transverse joint spacing on performance is evaluated by considering sections CA 1-1 and 1-3. The survey showed that faulting is much lower on the shorter-jointed sections, as is transverse cracking. There is little difference in the percent of joints spalled. However, these findings are not borne out when the replicate sections are considered. Section CA 1-2 (short-jointed) does display some transverse cracking (8 percent of slabs), but this is still less than CA 1-4. Overall, the shorter joint spacing does not appear to provide significant benefits over the longer jointed section. Although the faulting is lower, there are more joints per mile, which may explain the higher roughness and lower PSR for the short-jointed sections.

The effect of base type is evaluated by comparing the performance of sections CA 1-3 and CA 1-7. The results here are somewhat ambiguous. In terms of transverse joint faulting, there is clearly less faulting on the sections with the LCB than on the sections with the CTB. However, the level of transverse cracking is somewhat higher on the section with LCB than on the section with CTB. Similar trends are exhibited by the replicate sections. These results, considered together, suggest that the stiffer base (LCB) is more effective at reducing faulting, but may contribute to the development of transverse cracking by leading to increased thermal curling stresses. This analysis may particularly be true for the longer slabs.

The performance of pavements constructed with high-strength concrete can be considered by comparing sections CA 1-3 and CA 1-9. The high strength section (CA 1-9) had higher levels of faulting and transverse cracking, but less longitudinal

Table 19. Summary of 1987 and 1992 outer lane performance data for CA 1.

			12-13-19-18-ft (3.6-4.0-5.8-5.5-m) Joints						5-8-11-7 ft (1.5-2.4-3.4-2.1 m) Joints			8.4-in (213-mm) CRCP		
			8.4-in (213-mm) JPCP			11.4-in (290-mm) JPCP			8.4-in (213-mm) JPCP			Long. Bars 0.56%	Long + Tr.Bars 0.56%	WWF 0.56%
			1987 Survey	1992 Survey	1992 Rep	1987 Survey	1992 Survey	1992 Rep	1987 Survey	1992 Survey	1992 Rep	1992 Survey	1992 Survey	1992 Survey
Age at Survey, years			16	21	21	16	21	21	16	21	21	21	21	21
ESAL's, millions			7.6	11.9	11.9	7.6	11.9	11.9	7.6	11.9	11.9	11.9	11.9	11.9
Normal Strength Concrete (5.5 bag)	CTB (4%)		CA 1-3	CA 1-3	CA 1-4	CA 1-5	CA 1-5	CA 1-6	CA 1-1	CA 1-1	CA 1-2	CA 1-11 / 12	CA 1-13 / 14	CA 1-15 / 16
		Faulting, in	0.10	0.08	0.10	0.11	0.11	0.11	0.06	0.05	0.07	—	—	—
		Det. Tr. Crks/mi	30	60	240	0	0	135	5	5	56	0 / 0	0 / 0	111 / 0
		% Slabs Cracked	9	18	53	0	0	34	1	1	8	—	—	—
		Long. Cracks, ft/mi	872	812	0	0	65	0	0	0	507	0 / 0	0 / 0	0 / 0
		% Joints Spalled	3	1	3	3	3	1	2	1	1	Fails/mi: 0 / 0	Fails/mi: 0 / 0	Fails/mi: 48 / 0
		PSR	3.0	3.3	3.3	2.7	3.2	3.5	2.9	3.3	2.7	3.4 / 3.3	3.4 / 3.6	3.5 / 3.8
		IRI, in/mi	—	111	157	—	141	166	—	129	210	99 / 119	94 / 93	141 / 94
	LCB (4 bag)		CA 1-7	CA 1-7	CA 1-8									
		Faulting, in	0.06	0.02	0.04									
		Det. Tr. Crks/mi	75	85	255									
		% Slabs Cracked	22	24	60									
		Long. Cracks, ft/mi	180	210	85									
		% Joints Spalled	9	9	6									
		PSR	2.7	3.1	3.5									
		IRI, in/mi	—	120	106									
High Strength Concrete (7.5 bag)	CTB (4%)		CA 1-9	CA 1-9	CA 1-10									
		Faulting, in	0.13	0.11	0.13									
		Det. Tr. Crks/mi	190	271	321									
		% Slabs Cracked	39	63	71									
		Long. Cracks, ft/mi	519	551	70									
		% Joints Spalled	3	3	0									
		PSR	2.5	3.1	2.7									
		IRI, in/mi	—	130	210									

All sections have a 5.4-in (137-mm) base and a 24-in (610-mm) aggregate subbase.
CA 1-5 = SHRP 67456, CA 1-14 = SHRP 67455

1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km

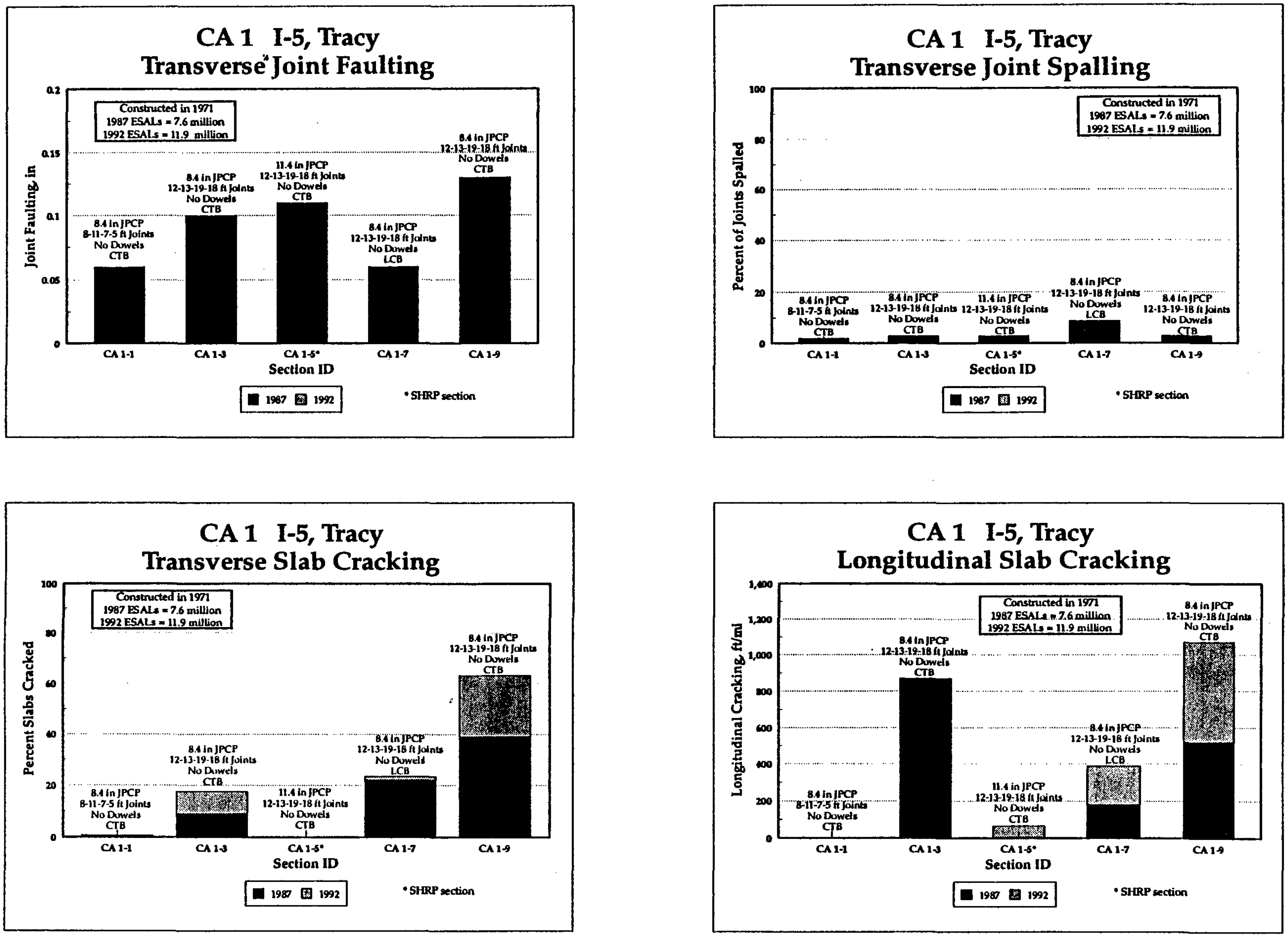


Figure 21. Performance summary for CA 1.

cracking. The high-strength replicate section (CA 1-10) had higher levels of faulting, transverse cracking, and longitudinal cracking, as well a lower PSR and higher IRI than the corresponding section (CA 1-4).

Three CRCP designs were included in the 1992 survey to compare the performance of pavement types. Table 19 shows that these sections, each with 0.56 percent longitudinal steel, are performing quite well. Only one section (CA 1-15) is exhibiting any deteriorated transverse cracks or pavement failures, and this section contains welded wire fabric (WWF) for reinforcement. The welded wire fabric (1-C) was in the form of mats with D-19 longitudinal bars spaced at 4 in (102 mm) welded to D-6 transverse wires spaced at 16 in (406 mm). The welded wire fabric apparently is not as effective as deformed bars in holding cracks tight in CRCP.

As observed from table 19, the magnitude of all of the distresses is higher on the replicate sections located in the southbound lanes (even-numbered sections) than on the sections in the northbound lanes (odd-numbered sections). This occurrence may be due to construction differences or to differences in traffic loadings from one direction to the other. Another explanation may be differences in elevation of the northbound and southbound lanes, which result in different support and drainage conditions.

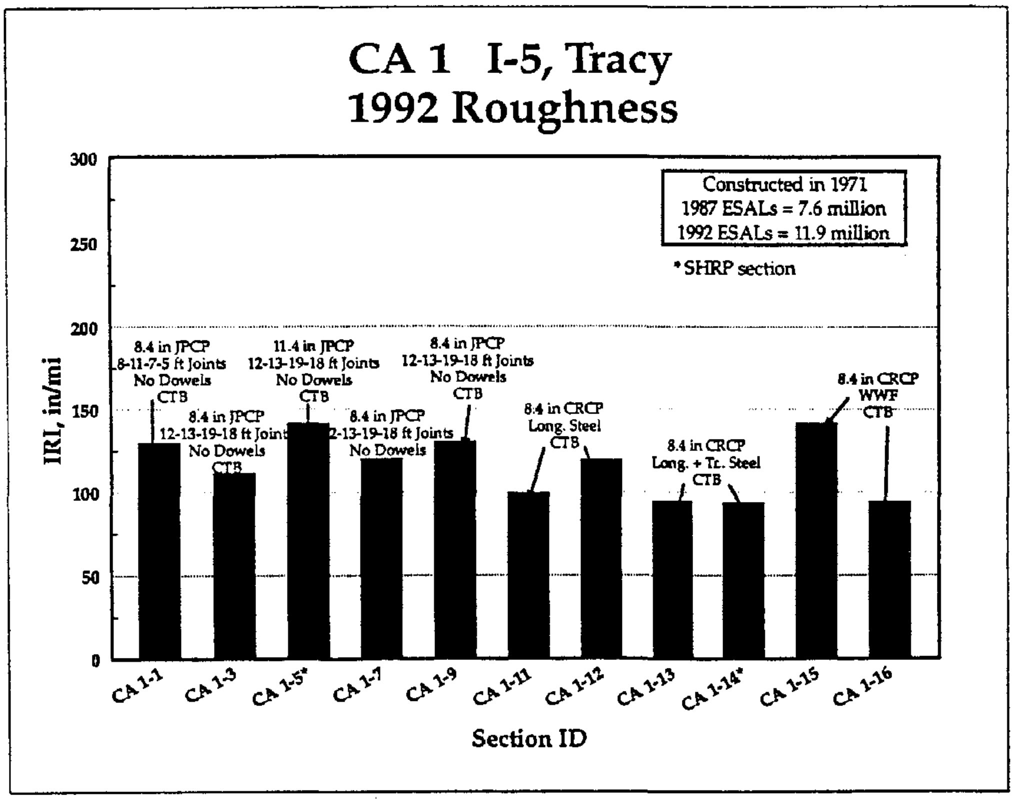
Roughness and Serviceability

Figure 22 illustrates roughness and serviceability values for the CA 1 sections. This figure shows that the smoothest-riding pavements are CA 1-3 (standard design) and CA 1-7 (LCB), while the roughest sections are CA 1-5 (thick slab) and CA 1-9 (high strength PCC). From the 1987 survey, the relative rideability of the sections has not changed.

Figure 22 also shows the superior riding surface provided by the CRCP sections. These designs are consistently smoother than the JPCP designs, even for the CRCP section exhibiting deteriorated transverse cracks and pavement failures (CA 1-15).

5-Year Performance Trends

The 5-year performance trends exhibited by these sections is illustrated in figure 23. None of the sections showed an increase in transverse joint faulting, and joint spalling remained virtually unchanged since 1987. Transverse cracking increased for most of the sections, with the biggest increase observed for CA 1-9 (high strength PCC). Longitudinal cracking also remained about the same for all sections over the 5-year period.



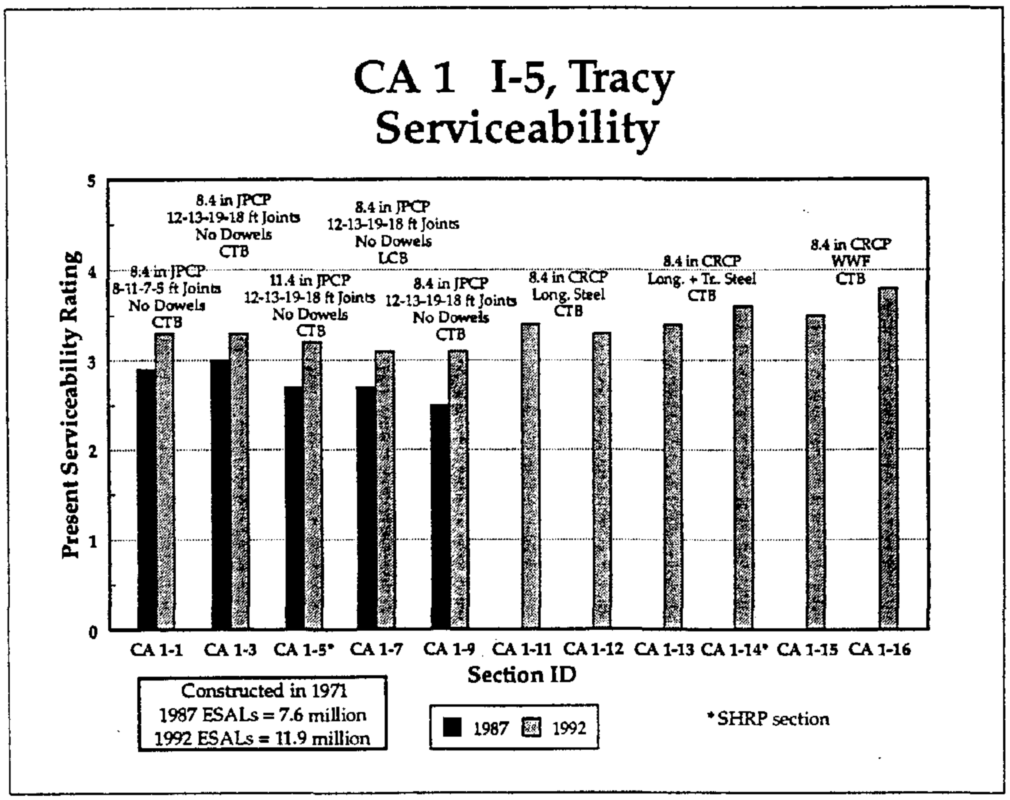


Figure 22. Roughness and serviceability for CA 1.

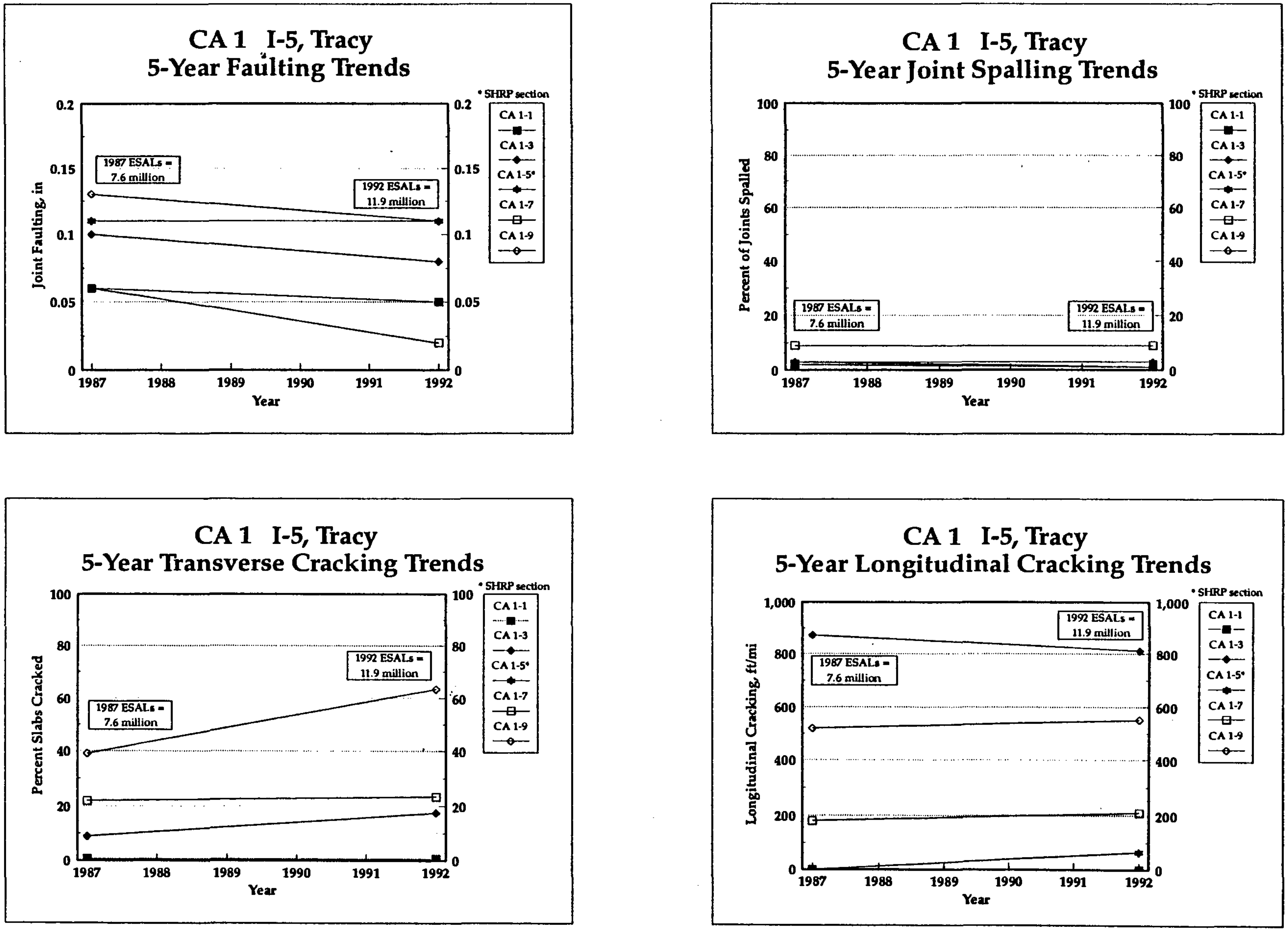


Figure 23. 5-year performance trends for CA 1.

Summary

The performance of the CA 1 sections is summarized below:

- The thickened slab (11.4-in [290-mm]) apparently reduced the amount of cracking but not the level of transverse joint faulting.
- As expected, the short-jointed section exhibited less faulting and transverse cracking than the section with conventional spacing. However, its roughness level was higher than the section with conventional spacing.
- The LCB was effective in reducing joint faulting, but apparently contributed to increased transverse cracking.
- The section with a higher cement content showed greater amounts of distress (faulting and cracking) than the other sections.
- The CRCP sections were exhibiting very good performance, with very little deterioration and overall rideability levels superior to the JPCP designs. The only CRCP section displaying deterioration was one containing WWF, which is probably not effective in holding transverse cracks tight in CRCP.
- Greater levels of deterioration were observed on the southbound lanes (even-numbered sections) than on the northbound lanes (odd-numbered sections).
- None of the sections displayed a significant increase in faulting or spalling since being evaluated in 1987, although transverse cracking did increase. In terms of rideability, the pavement sections have not deteriorated since 1987.

California 2 (I-210, Los Angeles)

In 1980, two different concrete pavement designs were constructed on I-210 near Los Angeles to evaluate the effect of base type on concrete pavement performance.[21] The only variable in these sections is base type and permeability; one section contains a permeable cement-treated base (PCTB), whereas the other section has an CTB. However, for the permeable base section, a thin layer of asphalt concrete was placed between the slab and the base, essentially causing the design to perform as if it had a nonpermeable base.

Common to both sections is an 8.4-in (213-mm) JPCP slab with transverse joints spaced at 12-13-19-18-ft (3.7-4.0-5.8-5.5-m) intervals. The transverse joints contain no dowel bars and are not sealed. The subgrade soil is an AASHTO A-4 material. The simplified design matrix for the project is shown in table 20.

Performance

The performance of these sections is shown in table 21. Although less faulting has occurred on the CTB section, the levels of faulting exhibited by both sections is excessive. In this case, the design of the permeable base was such that it could not

Table 20. Experimental design matrix for CA 2.

	Base Type	
	7.8-in (198-mm) Dense AC/PCTB (6-8% cement)	5.4-in (137-in) CTB (5% cement)
8.4-in (213-mm) JPCP 12-13-19-18-ft (3.7-4.0-5.8-5.5-m) Joints	CA 2-2	CA 2-3

CA 2-2 has 3-in (76-mm) aggregate subbase.
CA 2-3 has 6-in (152-mm) aggregate subbase.

Table 21. Summary of 1987 and 1992 outer lane performance data for CA 2.

		Base Type			
		7.8-in (198-mm) Dense AC/PCTB (6-8% cement		5.4-in (137-mm) CTB (5% cement)	
		1987 Survey	1992 Survey	1987 Survey	1992 Survey
Age at Survey, years		7	12	7	12
ESAL's, millions		4.4	9.1	4.4	9.1
8.4-in (213-mm) JPCP 12-13-19-18-ft (3.7-4.0-5.8-5.5-m) Joints		CA 2-2	CA 2-2	CA 2-3	CA 2-3
	Faulting, in	0.11	0.16	0.11	0.13
	Det. Tr. Crks/mi	0	10	290	311
	% Slabs Cracked	0	3	66	68
	Long. Cracks, ft/mi	0	0	0	0
	% Joints Spalled	0	3	0	15
	PSR	3.8	4.0	4.1	3.9
	IRI, in/mi	—	137	—	116

CA 2-2 has 3-in (76-mm) aggregate subbase.
CA 2-3 has 6-in (152-mm) aggregate subbase.

1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km

effectively remove moisture to reduce the development and progression of joint faulting. However, the section placed on CTB is exhibiting significantly more transverse cracking than the section placed on AC/PCTB. The stiffness of the CTB apparently is contributing to the development of the cracking. The CTB is also exhibiting slighter greater amounts of joint spalling.

Since 1987, both sections have developed additional transverse cracking, faulting, and joint spalling. CA 2-2 particularly has shown a significant increase in joint faulting. The roughness and serviceability values are roughly comparable for both sections, with no significant changes since 1987.

California 6 (RT. 14, Solemint)

Two separate concrete pavement sections, located on RT. 14 near Solemint in the greater Los Angeles area, were evaluated under this study. One section, constructed in 1971, is a 9-in (229-mm) JPCP section that contains transverse joints spaced at 12-13-19-18-ft (3.7-4.0-5.8-5.5-m) intervals and is constructed over a CTB. An adjacent section, constructed in 1980, is built on a permeable asphalt-treated base (PATB) and includes transverse joints spaced at 12-13-15-14-ft (3.7-4.0-4.6-4.3-m) intervals. While only the former section was surveyed in 1987, both sections were evaluated in 1992.

The design and performance of these sections are given in table 22. Section 6-1 is exhibiting significant joint faulting, but little structural deterioration (cracking or spalling). The condition has changed little since the evaluation in 1987.

Table 22. Design and outer lane performance data for CA 6.

	CA 6-1		CA 6-2
Pavement Type	JPCP		JPCP
Year Built	1971		1980
Thickness, in	9-in (229-mm)		9-in (229-mm)
Joint Spacing, ft	12-13-19-18 (3.7-4.0-5.8-5.5-m)		12-13-15-14 (3.7-4.0-4.6-4.3-m)
Dowel Diameter, in	None		None
Base Type	5.4-in (137-mm) CTB (4% cement)		4.2-in (107-mm) PATB (2%, AR-4000)
Subgrade	A-2-4		A-2-4
Performance Data	1987 Survey	1992 Survey	1992 Survey
Age at Survey, years	16	21	13
ESAL's, millions	8.6	13.3	9.8
Faulting, in	0.15	0.14	0.05
Deteriorated Tr. Crks/mi	10	10	0
% Slabs Cracked	3	4	0
Long. Cracks, ft/mi	0	0	0
% Joints Spalled	3	3	0
PSR	3.4	3.5	3.8
IRI, in/mi	—	173	170

CA 6-1 has a 24-in aggregate subbase.
CA 6-2 has reinforcing fabric, 1.8 in of ATB, and 33 in of aggregate beneath the ATB.

1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km

The performance of 6-2, being much newer than 6-1 and subjected to fewer traffic loadings, is quite good. No structural deterioration has occurred, although some faulting has developed and the roughness level appears to be somewhat high. Although structurally this pavement appears to be performing well, research conducted by the California Department of Transportation indicates that significant stripping of the PATB has occurred such that the aggregate is in a loose, unbound state.[23] Faulting and movement of the unbound aggregate was also noted. This condition is expected to adversely affect the future performance of this pavement.

California 7 (I-5, Sacramento)

Constructed in 1979 on I-5 near Sacramento, this single section includes a 10.2-in (259-mm) JPCP slab placed on a CTB. The joints are randomly spaced at 12-13-19-18-ft (3.7-4.0-5.8-5.5-m) intervals, skewed, and are not doweled. The AASHTO A-7 subgrade is lime stabilized to a depth of 5.4 in (137 mm). This project is one of the earliest projects incorporating California's then-current drainage design.

The drainage design consists of a 12-in (305-mm) layer of PATB beneath the asphalt shoulder. Longitudinal edge drains (1.5-in [38-mm] diameter) are located 15.6 in (396 mm) below the pavement surface (and within the PATB). Drainage fabric was placed at the PATB-subgrade interface to guard against migration of fines.

The design and performance of this section is summarized in table 23. This pavement has undergone very little change in performance over the past 5 years. All distresses are at about the same levels in 1992 as they were in 1987. The faulting is fairly low for a nondoweled pavement, which might illustrate the benefit of the drainage design and the stabilized base. The amount of longitudinal cracking that has developed in CA 7 is excessive, but this is believed to be due to longitudinal joint construction practices. In terms of roughness and serviceability, the pavement has lost some rideability since 1987 and is exhibiting a significant level of roughness.

California 8 (U.S. 101, Thousand Oaks)

This 10.2-in (259-mm) JPCP section is constructed with widened outer lanes. Built in 1983 on U.S. 101 near Thousand Oaks, the slab rests on a 5.4-in (137-mm) thick ATB course and an AASHTO A-7 subgrade material. The pavement has skewed, random joints (12-13-15-14-ft [3.7-4.0-4.6-4.3-m] intervals) and contains longitudinal edge drains.

The design and performance data for this section is summarized in table 23. There is little difference in any of the distresses from the 1987 survey to the 1992 survey, although some joint spalling has developed. Overall, however, this pavement is performing quite well.

California 11 (I-5, Thornton)

Built in 1979, this project is located on I-5 near Thornton. The pavement is an 8.4-in (213-mm) JPCP constructed on a lean concrete base (LCB). The joints do not contain dowel bars and are spaced at 12-13-19-18-ft (3.7-4.0-5.8-5.5-m) intervals.

Table 23. Design and outer lane performance data for CA 7 and CA 8.

	CA 7		CA 8	
Pavement Type	JPCP		JPCP	
Year Built	1979		1983	
Thickness, in	10.2 (259-mm)		10.2 (259-mm)	
Joint Spacing, ft	12-13-19-18 (3.7-4.0-5.8-5.5-m)		12-13-15-14 (3.7-4.0-4.6-4.3-m)	
Dowel Diameter, in	None		None	
Base Type	5.4-in (137-mm) CTB (5% cement)		5.4-in (137-mm) ATB (3%, AR-4000)	
Subbase	5.4-in (137-mm) lime stabilized		9-in (229-mm) aggregate	
Subgrade	AASHTO A-7		AASHTO A-7	
Performance Data	1987 Survey	1992 Survey	1987 Survey	1992 Survey
Age at Survey, years	8	13	4	9
ESAL's, millions	10.5	19.6	5.3	9.1
	CA 7	CA 7	CA 8	CA 8
Faulting, in	0.06	0.05	0.04	0.05
Deteriorated Tr. Crks/mi	119	119	0	0
% Slabs Cracked	32	32	0	0
Long. Cracks, ft/mi	2060	2025	0	0
% Joints Spalled	0	0	0	17
PSR	3.8	3.3	3.8	4.0
IRI, in/mi	—	134	—	148

Both sections contain longitudinal edge drains.
CA 8 has a 14-ft outside lane.

1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km

The design and performance data for this single section are summarized in table 24. The faulting is very low, although the slab cracking is higher than desired. The low IRI indicates that this distress is not having an adverse effect on the rideability of the pavement.

Performance Summary of Projects in Wet-Freeze Zone

In the wet-freeze environmental region, experimental concrete pavement sections are included from nine States and one Province: Illinois, Michigan, Missouri, New York, Ohio, Ontario, Pennsylvania, New Jersey, West Virginia, and Wisconsin. A total of 25 experimental and single-section projects are included in this region. Of all the environmental regions, this region contains the most projects and is the best represented.

Table 24. Design and outer lane performance data for CA 11.

	CA 11
Pavement Type	JPCP
Year Built	1979
Thickness, in	8.4 (213-mm)
Joint Spacing, ft	12-13-19-18 (3.7-4.0-5.8-5.5-m)
Dowel Diameter, in	0.00
Base Type	5.4-in (137-mm) LCB (10% cement)
Subbase	6-in (152-mm) lime stabilized
Subgrade	A-7
Performance Data	1992 Survey
Age at Survey, years ESAL's, millions	13 19.0
Faulting, in Deteriorated Tr. Crks/mi % Slabs Cracked Long. Cracks, ft/mi % Joints Spalled PSR IRI, in/mi	0.02 60 18 0 0 3.6 69

CA 11 = SHRP 63042.

1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km

Climatic indices for the region include a range of 25 to 1000 for the Corps of Engineers Freezing Index, a range of 30 to 60 for the Thornthwaite Moisture Index, and an annual precipitation range of 30 to 43 in (762 to 1092 mm). The highest average monthly maximum temperature ranges from 80 °F (27 °C) to 86 °F (30 °C). The lowest average monthly minimum temperature ranges from 10 °F (-12 °C) to 25 °F (-4 °C).

Illinois 1 (U.S. 50, Carlyle)

This project, constructed in 1986, is located approximately 20 mi (32.2 km) south of I-70 on U.S. 50, near Carlyle. Twenty-nine pavement sections are included in the project, consisting of CRCP, JRCP, and JPCP designs of varying thicknesses, sections with and without underdrains, sections with and without edge joint seals, and sections with different joint spacings (JRCP only). The purpose of this experimental project is to calibrate theoretical pavement responses (stress, strain, and deflection) of various pavement designs to those actually measured on inservice pavements.[24]

For the current study, only seven sections were selected for evaluation. Four sections are JRCP designs, each of which is 8.5-in (216-mm) thick and constructed on a 4-in (102-mm) LCB. One section is a conventional 40-ft (12.2-m) JRCP design, whereas the other three are "hinge joint" JRCP designs.

In the hinge joint designs, conventional doweled joints (1.5-in [38-mm] epoxy-coated dowels) are located at each end of the 40-ft (12.2-m) slab, with one or two intermediate joints placed within the slabs. These intermediate joints are sawed as conventional joints, but are located where additional reinforcing steel has been placed. The hinge joints contain 36 in (914 mm) long, epoxy-coated, No. 6 (19-mm) bars spaced at 18-in (457-mm) centers. The concept is to force transverse cracks (which are expected to occur in a JRCP design) to develop at the sawed joints, where the additional reinforcing steel will hold the sawed joint tight. The conventional JRCP design contains 0.13 percent reinforcing steel, whereas the hinged JRCP design contains 0.29 percent reinforcing steel. Some of the hinged sections were constructed without reinforcing mesh. However, since the reinforcing mesh was only placed between the joints and not through the joints, the effective steel content at the hinge joints is 0.29 percent whether or not reinforcing mesh was provided.

In addition to the JRCP sections, three CRCP sections were included for evaluation. These sections included slabs that are 7, 8, and 9 in (178, 203, 229 mm) thick. The selected CRCP sections are all placed on LCB, include longitudinal underdrains, and contain no edge joint seal. Longitudinal steel contents are 0.70, 0.73, and 0.72 percent of the cross sectional area for the 7-, 8-, and 9-in (178-, 203-, and 229-mm) slabs, respectively.

The IL 1 sections were all constructed on 4-in (102-mm) LCB without a subbase. The soils at the test site range from silty loams to clays, with the predominant soil type an AASHTO A-4 subgrade. Tied concrete shoulders were also installed after the construction of the mainline pavement.

Table 25 summarizes the selected sections from IL 1. Sections 1-13 and 1-14 have a hinge at 20 ft (6.1 m), but section 1-13 has mesh reinforcement and 1-14 does not. Section 1-15 has hinges at 13.3 ft (4.1 m), but no mesh. Section 1-16, which has mesh reinforcement but no hinge joint, is included to provide a comparison between the conventional design and the hinge joint design. Sections 1-1 and 1-2 are also included to allow performance comparisons of adjacent CRCP and JRCP sections.

Traffic Loadings

This pavement, a two-lane roadway, currently carries approximately 4,900 vehicles per day, including 20 percent heavy trucks. It is estimated to have sustained approximately 1.7 million 18-kip (80-kN) ESAL applications.

Table 25. Summary of sections on IL 1.

		Underdrains No Edge Joint Seal
7-in (178-mm) CRCP (0.70% Steel)		1-9
8-in (203-mm) CRCP (0.73% Steel)		1-2
9-in (229-mm) CRCP (0.72% Steel)		1-1
8.5-in (216-mm) JRCP 40-ft (12.2-m) Slabs 1.5-in (38-mm) Dowels	20-ft (6.1-m) hinge (wire mesh)	1-13
	20-ft (6.1-m) hinge (no mesh)	1-14
	13.3-ft (4.1-m) hinge (no mesh)	1-15
	Conventional, No hinge (wire mesh)	1-16

All sections contain a 4-in (102-mm) LCB, no subbase, and tied PCC shoulders.
Conventional JRCP design has 0.13% steel.
Hinged JRCP design has 0.29% steel through hinge joints.

Performance

The performance of the IL 1 sections is summarized in table 26 and in figure 24. In this project, the advantage of the hinge design over the design without hinges is clearly demonstrated in terms of the amount of deteriorated transverse cracks. None of the hinged joint sections are displaying any deteriorated transverse cracks, whereas the conventional 40-ft (12.2-m) JRCP design is displaying extensive medium- and high-severity transverse cracks. Within the hinge joint design, no statements can be made at this time regarding the relative merits of hinge joint spacing or the inclusion of mesh reinforcement.

Some longitudinal cracking had occurred in these sections, particularly in the conventional 40-ft (12.2-m) JRCP design. This cracking is believed to be the result of a construction problem. All sections are exhibiting virtually no joint faulting and only a minimal amount of spalling.

The CRCP sections are performing very well, exhibiting no deteriorated cracks and no failures (punchouts or patches of punchouts). Crack spacings on these pavements were typically between 3 and 3.5 ft (0.9 and 1.1 m).

Table 26. Summary of 1992 performance data for IL 1.

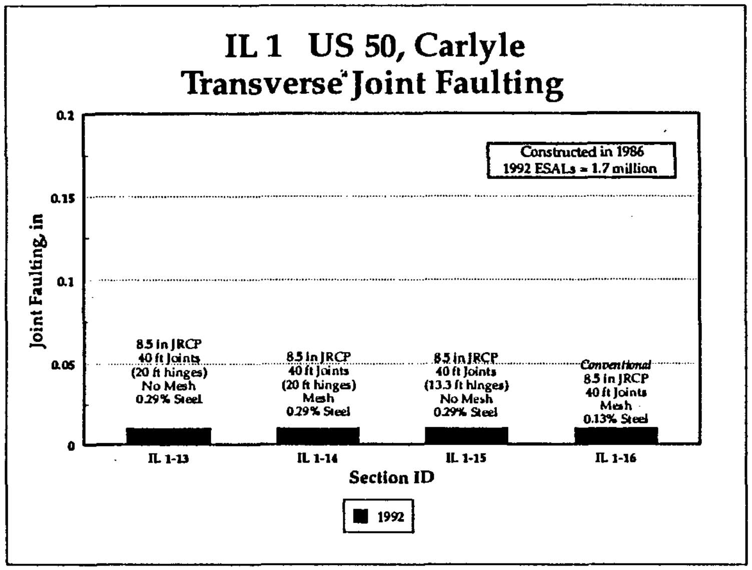
		Underdrains, No Edge Joint Seal			
		Conventional No Hinge Wire Mesh	20-ft (6.1-m) Hinge No Mesh	20-ft (6.1-m) Hinge Wire Mesh	13.3-ft (4.1-m) Hinge No Mesh
Age at Survey, years		6	6	6	6
ESAL's, millions		1.7	1.7	1.7	1.7
7-in (178-mm) CRCP (0.7% steel)		IL 1-9			
	Det. Tr. Crks/mi	0			
	Long. Cracks, ft/mi	0			
	Failures/mi	0			
	PSR	—			
	IRI, in/mi	123			
8-in (203-mm) CRCP (0.73% steel)		IL 1-2			
	Det. Tr. Crks/mi	0			
	Long. Cracks, ft/mi	0			
	Failures/mi	0			
	PSR	—			
	IRI, in/mi	114			
9-in (229-mm) CRCP (0.72% steel)		IL 1-1			
	Det. Tr. Crks/mi	0			
	Long. Cracks, ft/mi	0			
	Failures/mi	0			
	PSR	—			
	IRI, in/mi	103			
8.5-in (216-mm) JRCP 40-ft (12.2-m) Joints 1.5-in (38-mm) Dowels		IL 1-16	IL 1-13	IL 1-14	IL 1-15
	Faulting, in	0.01	0.01	0.01	0.01
	Det. Tr. Crks/mi	129	0	0	0
	Long. Cracks, ft/mi	1072	0	132	236
	% Joints Spalled	8	4	10	3
	PSR	—	—	—	—
	IRI, in/mi	119	153	173	168

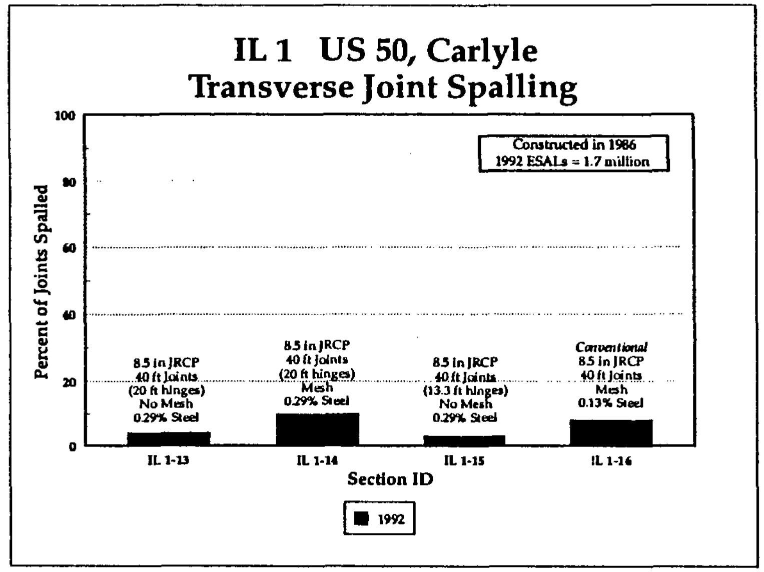
All sections contain a 4-in LCB, no subbase, and tied PCC shoulders.
Conventional JRCP design has 0.13% steel.
Hinge JRCP design has 0.29% steel.

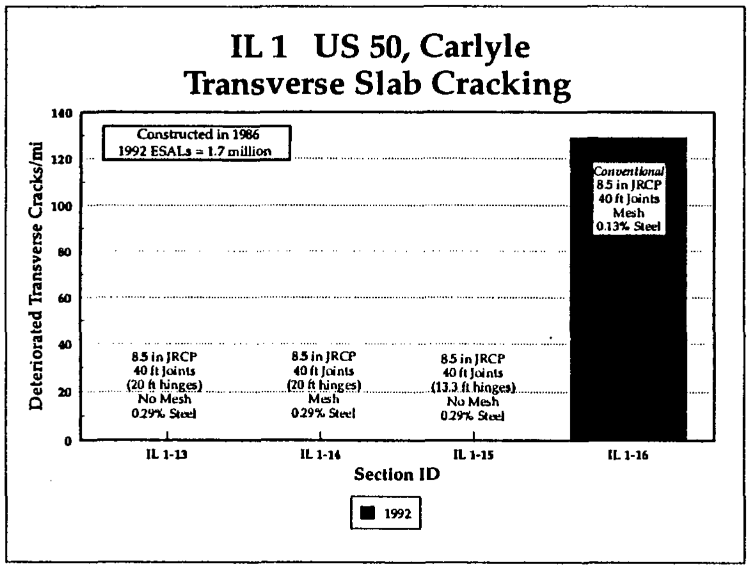
1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km

Roughness and Serviceability

Figure 25 illustrates the roughness measurements recorded for the IL 1 sections. With one exception, the roughness levels for the CRCP sections are much less than the JRCP designs. The lone exception is IL 1-16 (the conventional 40-ft [12.2-m] design), whose roughness levels were comparable to the CRCP sections. This result is somewhat surprising since IL 1-16 displayed a great deal of deteriorated transverse cracks. One explanation could be that the increased number of transverse joints on the hinge joint sections, even though not severely faulted, contributes to an increase in roughness. Another explanation could be related to the initial construction quality.







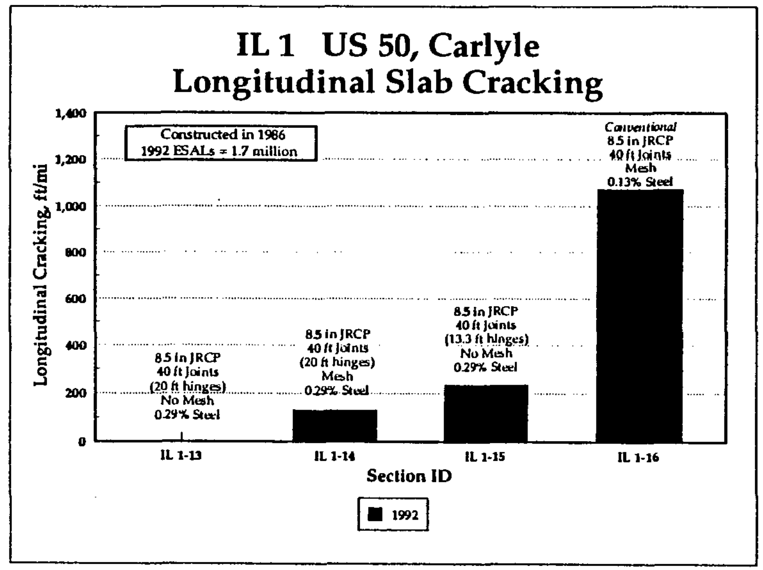


Figure 24. Performance summary for IL 1.

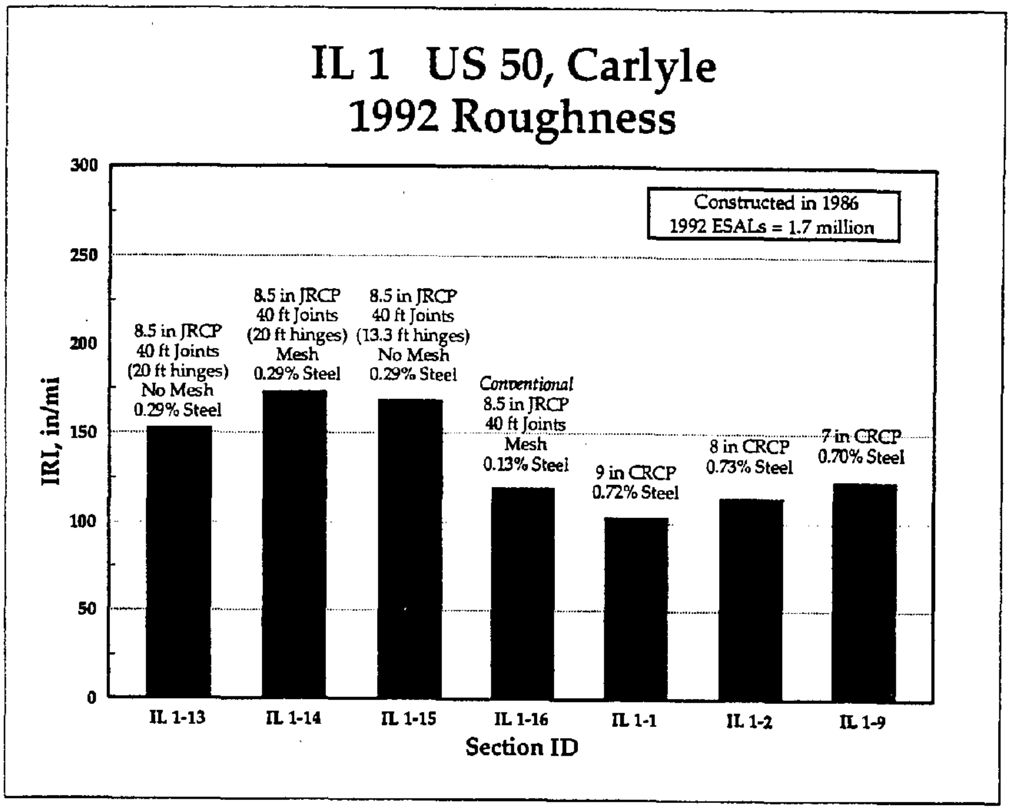


Figure 25. Roughness for IL 1.

Summary

The performance of the IL 1 sections is summarized below:

- The JRCP sections with the hinged joint design are performing better than the conventional JRCP design with 40-ft (12.2-m) joint spacing. The hinged joint design displays no deteriorated transverse cracking, whereas nearly every slab in the conventional design shows deteriorated transverse cracks. No difference in performance between the various hinge joint designs could be determined.

- Some longitudinal cracking was observed on the sections, but this is believed to be the result of construction practices.

- The CRCP sections are performing very well, exhibiting no structural deterioration. With the exception of the conventional JRCP design, the rideability of the CRCP sections was superior to that of the JRCP designs.

Illinois 2 (U.S. 20, Freeport)

The project, constructed in 1986, is located on U.S. 20 near Freeport. Like IL 1, this project was constructed to calibrate theoretical pavement responses (stress, strain, and deflection) of various pavement designs to those actually measured on inservice pavements.[24]

Four pavement sections are included in the project, each a 10-in (254-mm) thick JRCP constructed over a 4-in (102-mm) LCB. However, the transverse joint spacing varies, ranging from 15 to 40 ft (4.6 to 12.2 m). In addition, three sections containing hinge joints (as described in IL 1) are also included. The hinge joints contain 36 in (914 mm) long, epoxy-coated, No. 6 (19-mm) bars spaced at 18-in (457-mm) centers. The soils at the test site are a silty loam material, classified as an AASHTO A-4 material. Tied concrete shoulders are also present, having been installed after the construction of the mainline pavement.

The primary sections of interest on this project are those containing the hinge joint design (sections 2-5, 2-6, and 2-7). Section 2-8 is included to provide a comparison between the conventional design and the hinge joint design. These sections are shown in table 27.

Table 27. Summary of sections on IL 2.

	10-in (254-mm) JRCP 4-in (102-mm) LCB			
	Conventional No Hinge Wire Mesh 0.11% Steel	20-ft (6.1-m) Hinge No Mesh 0.25% Steel	20-ft (6.1-m) Hinge Wire Mesh 0.25% Steel	13.3-ft (4.1-m) Hinge No Mesh 0.25% Steel
40-ft (12.2-m) Joints 1.5-in (38-mm) Dowels	IL 2-8	IL 2-5	IL 2-6	IL 2-7

Traffic Loadings

Recent traffic data indicate the two-way ADT for this 2-lane roadway is 4,800 vehicles per day, including 16 percent heavy trucks. Through 1992, this pavement is estimated to have sustained about 1.3 million 18-kip (80-kN) ESAL applications.

Performance

The performance data are provided in table 28 and displayed graphically in figure 26. Similar to the IL 1 sections, a distinct difference in performance is noticed between the sections with and without the hinge joint. The control section without hinge joints has developed deteriorated transverse cracks in many of the 40-ft

Table 28. Summary of 1992 performance data for IL 2.

		10-in (254-mm) JRCP 4-in (102-mm) LCB			
		Conventional No Hinge Wire Mesh 0.11% Steel	20-ft (6.1-m) Hinge No Mesh 0.25% Steel	20-ft (6.1-m) Hinge Wire Mesh 0.25% Steel	13.3-ft (4.1-m) Hinge No Mesh 0.25% Steel
Age at Survey, years		6	6	6	6
ESAL's, millions		1.3	1.3	1.3	1.3
		IL 2-8	IL 2-5	IL 2-6	IL 2-7
40-ft (12.2-m) Joints	Faulting, in	0.00	0.01	0.02	0.03
	Det. Tr. Crks/mi	42	0	0	0
	Long. Cracks, ft/mi	0	0	0	0
1.5-in (38-mm) Dowels	% Joints Spalled	8	2	12	3
	PSR	—	—	—	—
	IRI, in/mi	131	121	127	96

1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km

(12.2-m) slabs, and the three sections containing hinge joints exhibit no such cracking. In addition, no section has longitudinal cracking, which may support the previous contention that this occurrence in the IL 1 sections is due to construction problems. The faulting exhibited by each section is virtually nonexistent. In addition, no significant difference in the amount of joint spalling is exhibited by these sections.

Roughness and Serviceability

Pavement roughness levels for these sections is shown in figure 27. This figure shows that the roughness of the hinge joint designs is less than that of the conventional design, although the differences are not significant. Interestingly, the smoothest-riding pavement is section 2-7, which contains the largest number of transverse joints. Again, this result may be related to the initial construction quality.

Summary

A summary of the performance of the IL 2 sections is provided below:

- The sections containing the hinge joint design are performing better than the conventional design in terms of the amount of deteriorated transverse cracking. However, there are no other significant differences in performance.
- No differences in performance exist between the different hinge joint designs.
- Roughness levels for each of the sections are not significantly different.

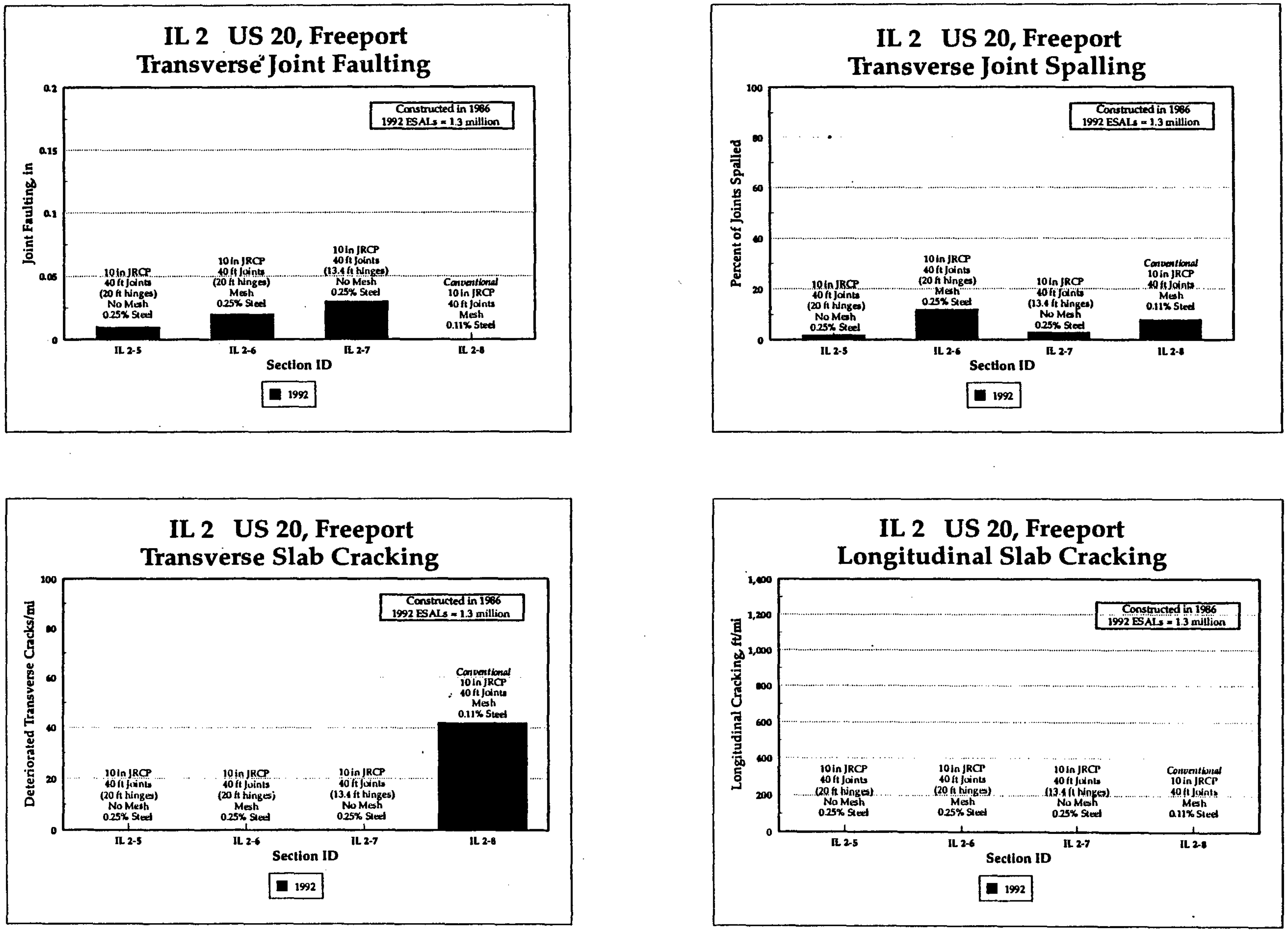


Figure 26. Performance summary for IL 2.

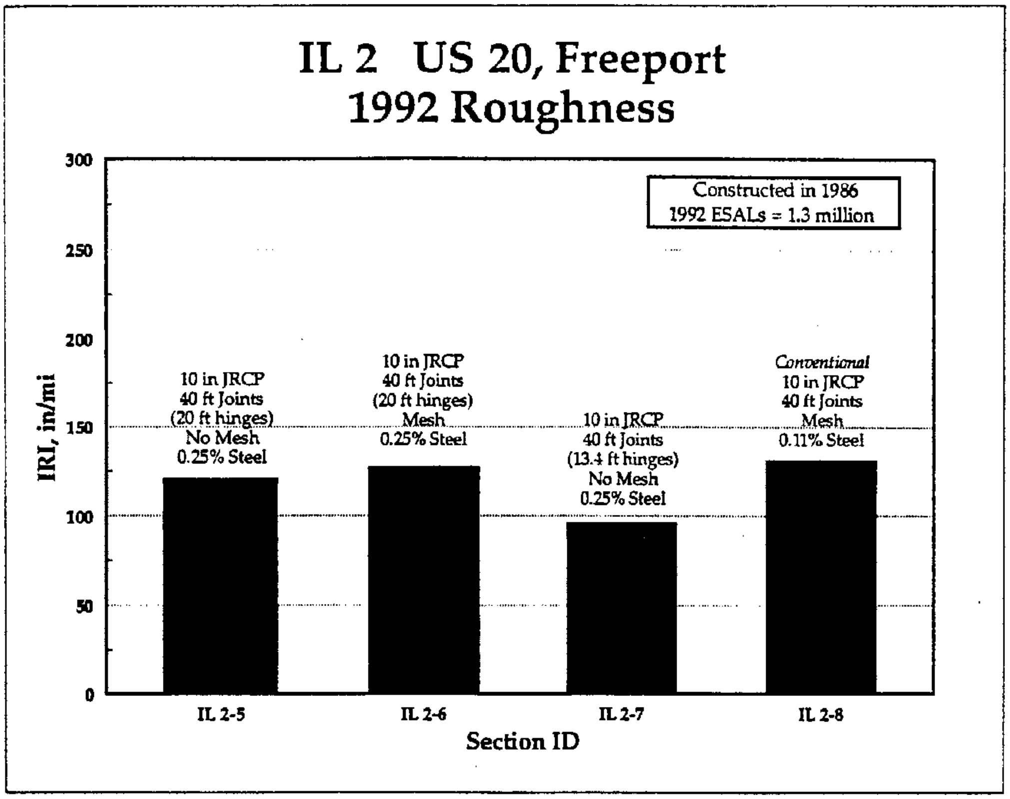


Figure 27. Roughness for IL 2.

Michigan 1 (U.S. 10, Clare)

Michigan 1 is an experimental project constructed on U.S. 10 near Clare. Built in 1975, the purpose is to evaluate the effects of base type, drainage, joint spacing, joint skew, and dowels on the performance of jointed concrete pavements.[25] The project includes both plain and reinforced jointed concrete pavements with 9-in (229-mm) slabs. One section (MI 1-25) contains a tied PCC shoulder, whereas the other sections contain a full-depth AC shoulder. A total of 25 sections were constructed, representing 7 different designs:

- JPCP on aggregate base with random joint spacing (12-13-17-16 ft [3.7-4.0-5.2-4.9 m]) and nonskewed, doweled joints—drained and nondrained sections (two designs).
- JPCP on permeable ATB (PATB) with random joint spacing (12-13-19-18 ft [3.7-4.0-5.8-5.5 m]) and skewed, nondoweled joints (one design).
- JPCP on ATB with random joint spacing (12-13-19-18 ft [3.7-4.0-5.8-5.5 m]) and skewed, nondoweled joints—drained and nondrained sections (two designs).
- JRCP on aggregate base with 71.2-ft (21.7-m) joint spacing and nonskewed, doweled joints—drained and nondrained sections (two designs).

The experimental design matrix for the sections surveyed on MI 1 is shown in table 29. Although French drains were retrofitted in 1981, it should still be possible to compare the performance of drained and nondrained sections. The subgrade for all sections is an AASHTO A-2-4.

Table 29. Experimental design matrix for MI 1.

	Drained		Nondrained	
	Skewed Joints	Nonskewed Joints	Skewed Joints	Nonskewed Joints
	No Dowels	1.25-in (32-mm) Epoxy-coated Dowels	No Dowels	1.25-in (32-mm) Epoxy-coated Dowels
9-in (229-mm) JPCP 12-13-17-16-ft (3.7-4.0-5.2-4.9-m) Jts 4-in (102-mm) AGG		MI 1-7a MI 1-7a5		MI 1-7b MI 1-7b5
9-in (229-mm) JPCP 12-13-19-18-ft (3.7-4.0-5.8-5.5-m) Jts 4-in (102-mm) PATB (2-3% of 85-100 pen)	MI 1-4a MI 1-4a10 MI 1-4a12			
9-in (229-mm) JPCP 12-13-19-18-ft (3.7-4.0-5.8-5.5-m) Jts 4-in (102-mm) ATB (6-8% of 250-300 pen)	MI 1-10a MI 1-10a3		MI 1-10b MI 1-25	
9-in (229-mm) JRCP (0.15% steel) 71.2-ft (21.7-m) Jts 4-in (102-mm) AGG		MI 1-1a MI 1-1a2		MI 1-1b MI 1-1b2

All sections constructed on a 10-in (254-mm) aggregate subbase and A-2-4 subgrade.
MI 1-25 has an adjacent PCC lane, others have an AC shoulder.

Traffic Loadings

The current two-way ADT for this four-lane divided roadway is 5,500 vehicles per day, which includes 9 percent heavy trucks. In 1987, the outer lane of this pavement had accumulated nearly 0.9 million 18-kip (80-kN) ESAL applications. Through 1992, the outer lane is estimated to have carried nearly 1.3 million ESAL applications.

Performance

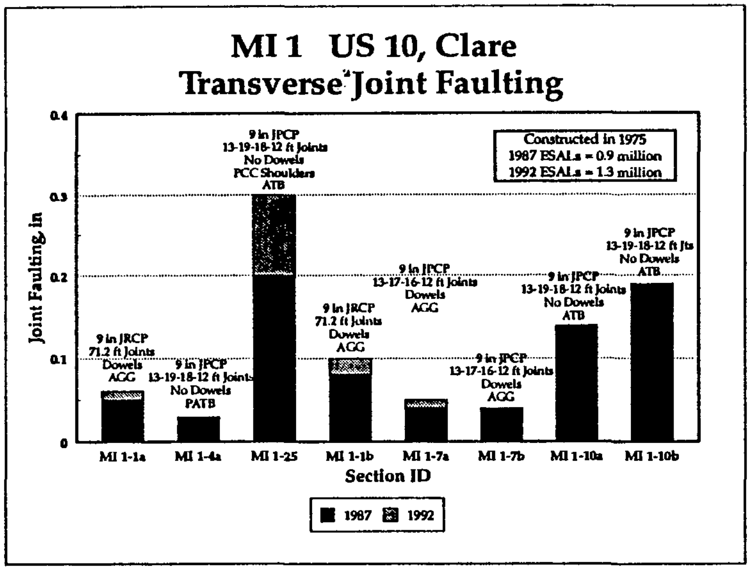
Table 30 summarizes the performance data for MI 1; this information is illustrated graphically in figure 28. This information clearly illustrates the poor performance exhibited by the pavements constructed on the dense-graded ATB. Not only did they exhibit the highest levels of faulting (0.13 in [3.3 mm] and higher), they also were the only sections to exhibit significant levels of transverse cracking and transverse joint

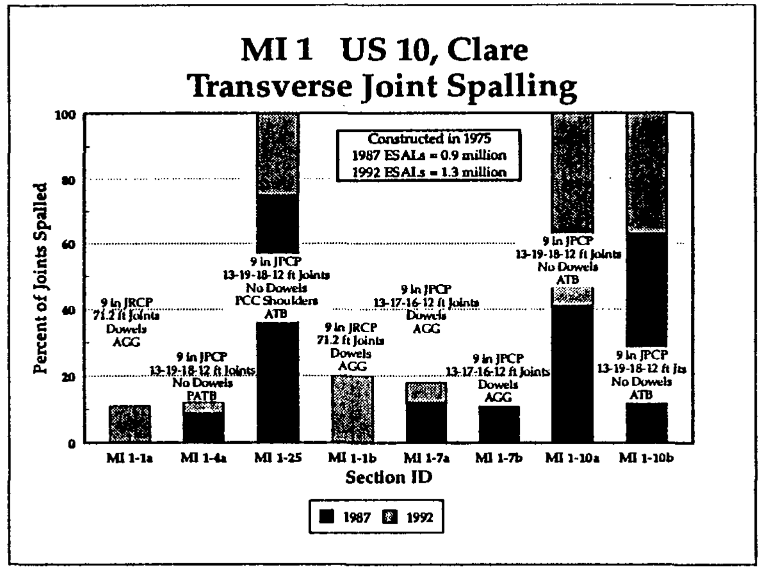
Table 30. Summary of 1987 and 1992 outer lane performance data for MI 1.

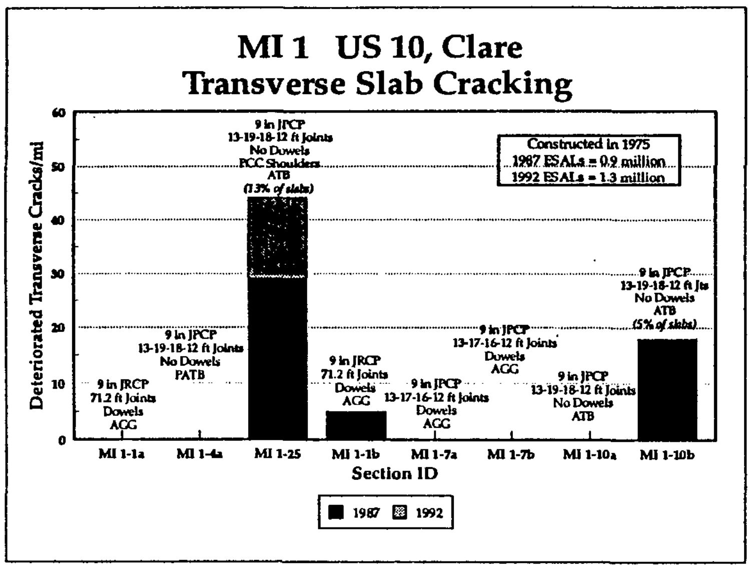
		Drained							Nondrained						
		Skewed Joints				Nonskewed Joints			Skewed Joints				Nonskewed Joints		
		No Dowels				1.25-in (32-mm) Epoxy-coated Dowels			No Dowels				1.25-in (32-mm) Epoxy-coated Dowels		
		1987 Survey	1992 Survey	1992 Rep.		1987 Survey	1992 Survey	1992 Rep.	1987 Survey		1992 Survey		1987 Survey	1992 Survey	1992 Rep.
Age at Survey, years		12	17	17		12	17	17	12		17		12	17	17
ESAL's, millions		0.9	1.3	1.3		0.9	1.3	1.3	0.9		1.3		0.9	1.3	1.3
9-in (229-mm) JPCP						1-7a	1-7a	1-7a5					1-7b	1-7b	1-7b5
	Faulting, in					0.04	0.05	0.05					0.04	0.03	0.06
12-13-17-16-ft Jts (3.7-4.0-5.2-4.9-m)	Det. Tr. Crks/mi					0	0	0					0	0	0
	% Slabs Cracked					0	0	0					0	0	0
	Long Cracks, ft/mi					0	0	0					0	0	0
	% Joints Spalled					12	18	0					11	11	20
4-in (102-mm) AGG	PSR					3.6	3.0	—					3.7	3.1	—
	IRI, in/mi					—	121	130					—	120	121
9-in (229-mm) JPCP		1-4a	1-4a	1-4a10	1-4a12										
	Faulting, in	0.03	0.03	0.02	0.02										
12-13-19-18-ft Jts (3.7-4.0-5.8-5.5-m)	Det. Tr. Crks/mi	0	0	0	0										
	% Slabs Cracked	0	0	0	0										
	Long Cracks, ft/mi	0	0	0	0										
4-in (102-mm) PATB (2-3% of 85-100 pen)	% Joints Spalled	9	12	6	0										
	PSR	3.9	3.8	—	—										
	IRI, in/mi	—	106	117	110										
9-in (229-mm) JPCP		1-10a	1-10a	1-10a3					1-10b	1-25	1-10b	1-25			
	Faulting, in	0.14	0.13	0.29					0.19	0.20	0.15	0.30			
12-13-19-18-ft Jts (3.7-4.0-5.8-5.5-m)	Det. Tr. Crks/mi	0	0	52					18	29	18	44			
	% Slabs Cracked	0	0	15					5	8	5	13			
	Long Cracks, ft/mi	0	0	0					0	0	0	0			
4-in (102-mm) ATB (6-8% of 250-300 pen)	% Joints Spalled	41	100	79					63	75	100	100			
	PSR	2.9	2.0	—					2.8	2.9	2.1	2.3			
	IRI, in/mi	—	161	203					—	—	197	247			
9-in (229-mm) JRCP						1-1a	1-1a	1-1a2					1-1b	1-1b	1-1b2
	Faulting, in					0.05	0.06	0.03					0.08	0.10	0.06
71.2-ft (21.7-m) Joints	Det. Tr. Crks/mi					0	0	32					5	5	0
	Long Cracks, ft/mi					0	0	0					0	0	0
	% Joints Spalled					0	11	0					0	20	33
4-in (102-mm) AGG	PSR					3.6	3.5	—					3.3	2.9	—
	IRI, in/mi					—	141	174					—	135	136

All sections contain a 10-in aggregate subbase and A-2-4 subgrade.
MI 1-25 has an adjacent PCC lane, others have an AC shoulder.

1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km







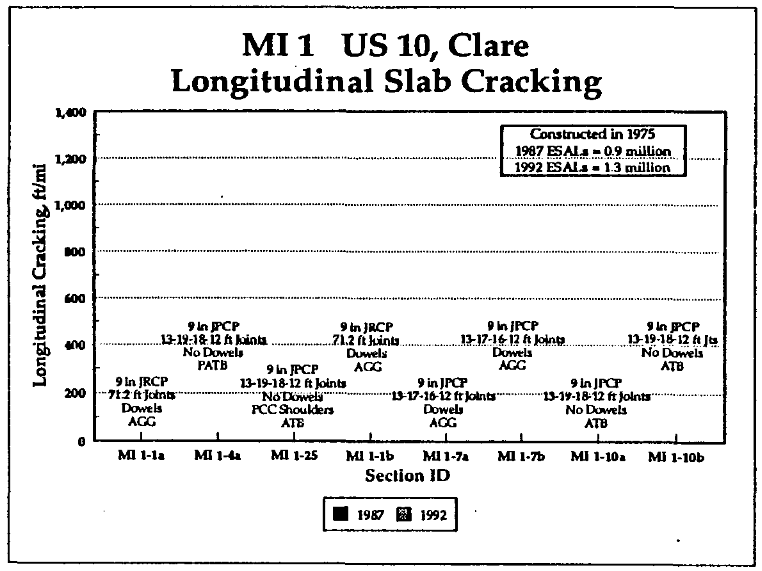


Figure 28. Performance summary for MI 1.

spalling. These ATB sections were constructed as a "bathtub" design with full-depth AC shoulders, which served to retain excess moisture within the pavement structure and resulted in the accelerated development of pavement distress.

Joint spalling on this project was caused by D-cracking, which was present throughout the project. However, the severity of the D-cracking (and hence the amount of spalling) was less on pavements constructed on either PATB or aggregate base course. This result is believed to be due to the drainage allowed by each of these base courses (the aggregate base course, even though dense-graded, allowed some degree of drainage). Sections constructed on the ATB apparently did not drain water very well because of the bathtub design and the impermeability of the ATB.

The nondoweled section constructed on PATB provided the best overall performance. It exhibited the least amount of joint faulting, displayed only minimal spalling, and showed no slab cracking. This result is surprising because 100 percent of the corners had voids, which may affect the long-term performance. A replicate section surveyed in the other direction exhibited similar performance. Both the long-jointed and short-jointed pavements placed on aggregate bases exhibited fairly good performance, although these sections all contained dowel bars.

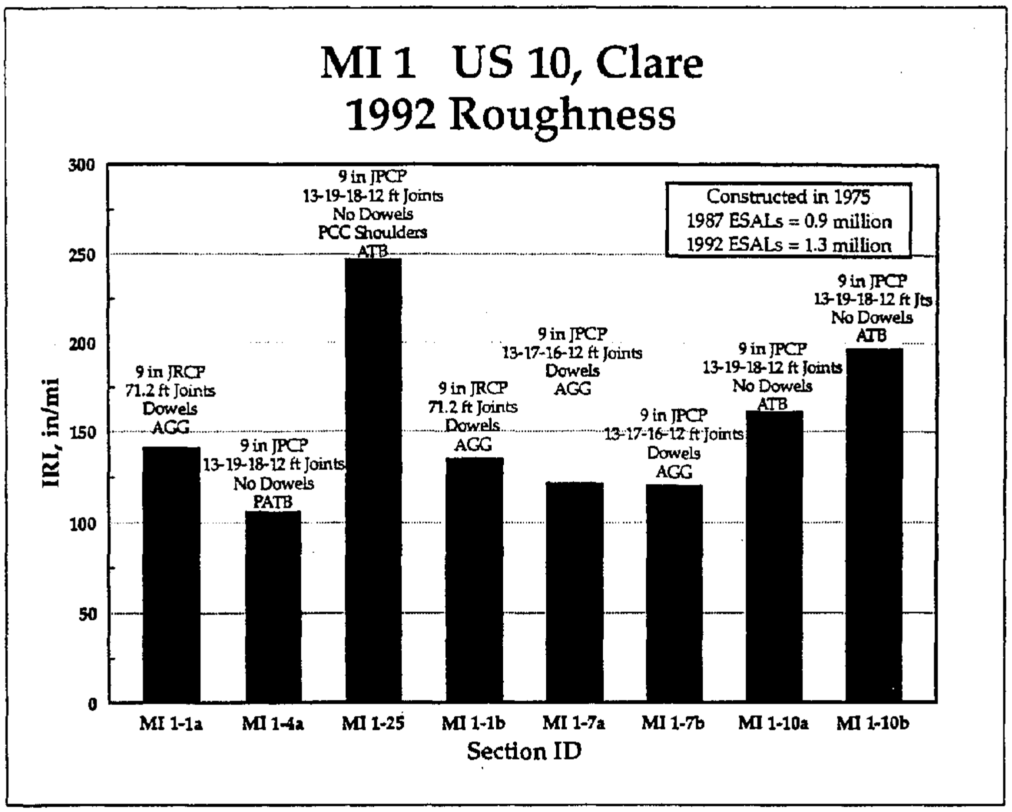
Direct comparisons between doweled and nondoweled sections cannot be made since the effect is confounded by base type. The nondoweled sections placed on ATB exhibited far greater levels of faulting than the doweled sections constructed on AGG. However, as previously noted, the nondoweled section placed on PATB exhibited faulting levels comparable to the doweled JPCP sections constructed on AGG.

The effect of drainage can be assessed by examining pavement sections with -a and -b suffixes. For the long-jointed JRCP designs, the drained pavement section (MI 1-1a) show less faulting and joint spalling than the nondrained pavement section (MI 1-1b). This is also true for the replicate sections. However, the effect of drainage is somewhat ambiguous in looking at the short-jointed JPCP designs (MI 1-7a and 1-7b), in that there is no clear advantage in the performance of the drained section.

In comparing the performance of pavement type, the long-jointed JRCP designs on aggregate base (MI 1-1a and 1-1b) are performing about the same as the short-jointed JPCP designs on aggregate base. Levels of faulting and spalling are approximately the same for each section. Because of confounding variables, the effect of joint spacing and of skewed joints on pavement performance cannot be determined.

Roughness and Serviceability

Roughness and serviceability for the MI 1 sections are shown in figure 29. This data appears to confirm that the sections constructed on ATB are performing the worst, as they are exhibiting the highest roughness levels and the lowest serviceability ratings. The smoothest-riding pavement is the section constructed on PATB, followed closely by the short-jointed pavements constructed on AGG.



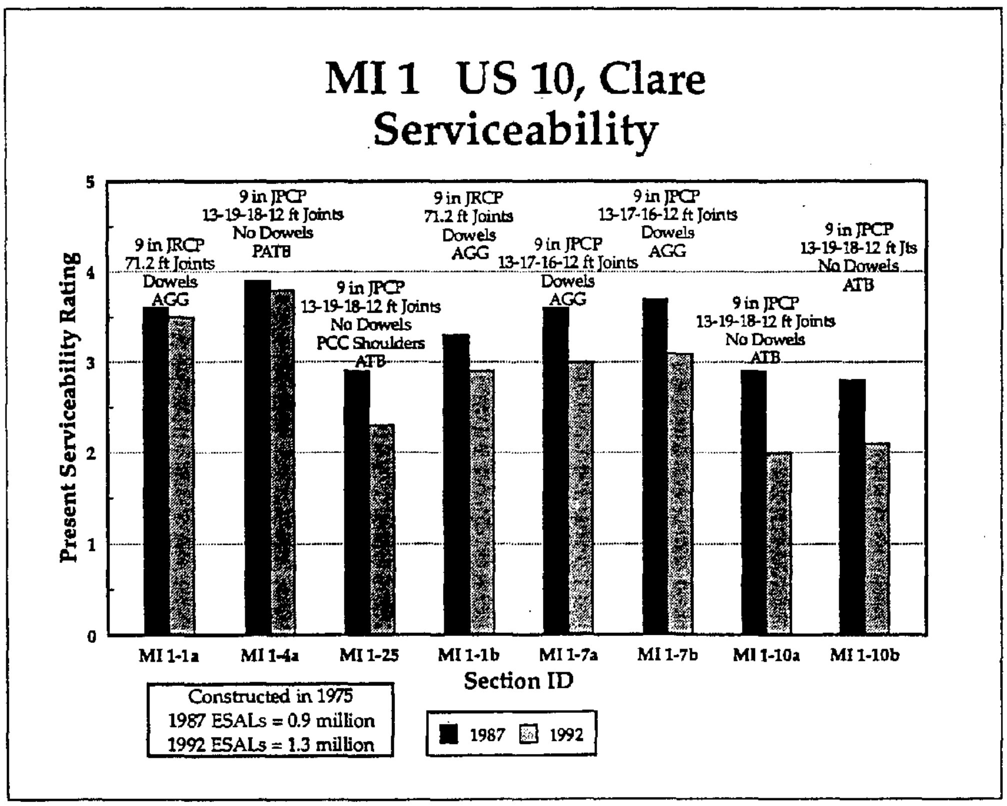


Figure 29. Roughness and serviceability for MI 1.

Over the past 5 years, the sections showing the largest decrease in serviceability are those constructed on ATB. The section on PATB and the long-jointed JRCP sections show the smallest decrease in serviceability over that time period.

5-Year Performance Trends

The 5-year performance trends for the key distress types are given in figure 30. This figure indicates that the pavements showing the largest increase in every distress category are those constructed on ATB. Most notably, the levels of spalling for each of the ATB sections increased to 100 percent. Section MI 1-25 is the only section to show a significant increase in faulting and transverse cracking.

Summary

The performance of the MI 1 project is summarized below:

- The pavement sections constructed on the dense-graded ATB showed the worst overall performance. These sections, which were constructed in a bathtub design (water trapped in the section), exhibited extremely high levels of faulting and joint spalling.

- D-cracking was prevalent throughout the project, but its severity was less on those pavements that allowed some degree of drainage. However, only a small amount of D-cracking was observed on the section constructed on PATB.

- The nondoweled section constructed on PATB performed the best of all the pavement sections. It exhibited faulting levels comparable to those displayed by doweled pavement sections constructed on AGG. However, 100 percent of the corner are showing voids, which could affect the long-term performance.

- Direct comparisons to evaluate the effect of dowels, joint spacing, and skewed joints were not possible, although the presence of dowels appeared to have a positive effect in reducing joint faulting.

- The positive effect of drainage was observed on the long-jointed JRCP sections, but not on the doweled, short-jointed JPCP sections.

- Similar performance was observed between the long-jointed JRCP and the short-jointed doweled JPCP sections (each constructed on AGG base).

- Roughness measurements and serviceability ratings confirm that the sections constructed on ATB are the worst-performing sections on the project. The section constructed on PATB is the smoothest-riding section.

- Over the last 5 years, the ATB sections, particularly MI 1-25, are showing the most significant increases in pavement distress.

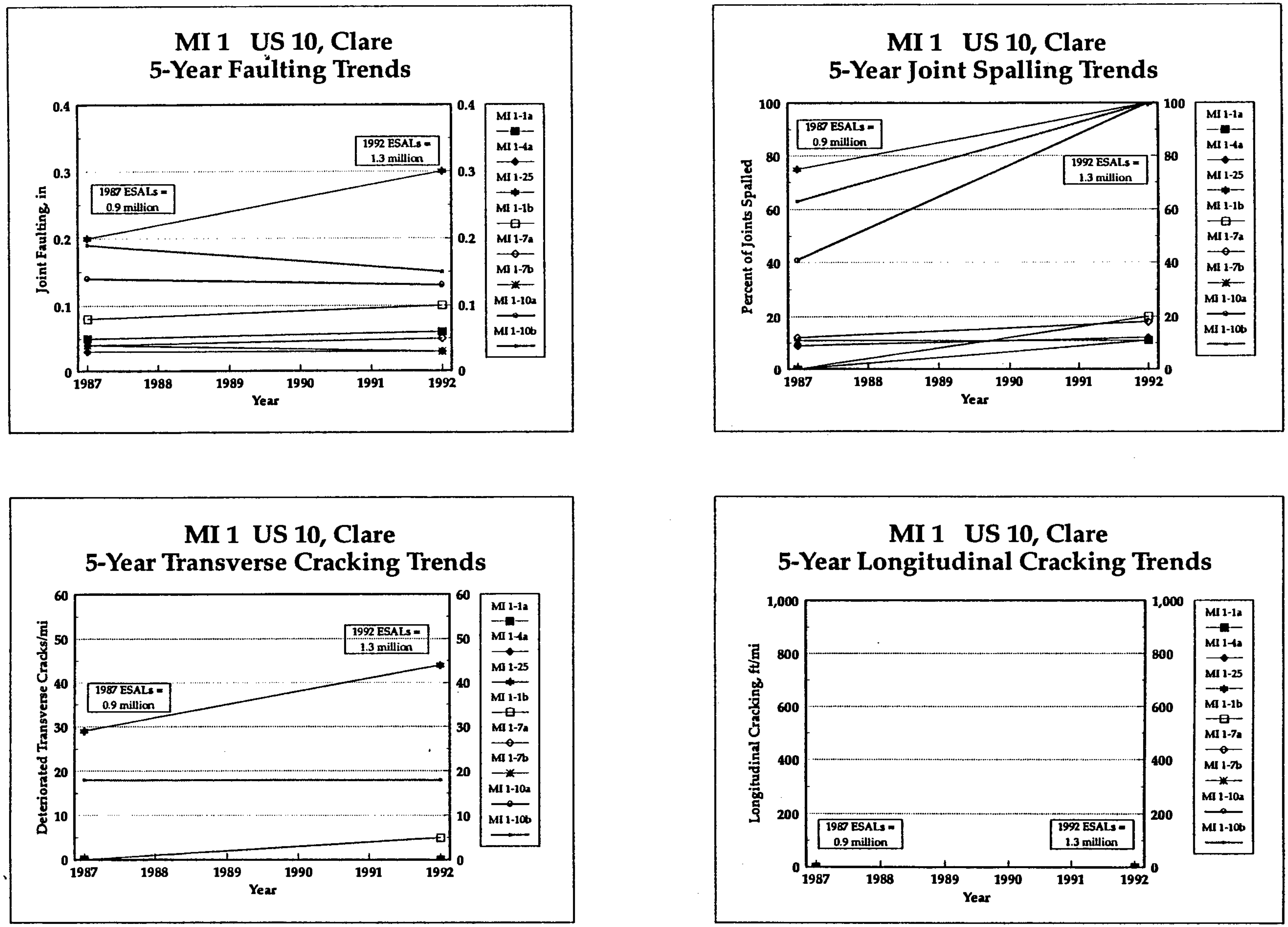


Figure 30. 5-year performance trends for MI 1.

Michigan 3 (I-94, Marshall)

Michigan 3, located on I-94 near Marshall, is a single section that was constructed in 1986 using recycled aggregate. MI 3 is included in this study because of its permeable aggregate base (PAGG) and its tied, reinforced concrete shoulders. The pavement is a 10-in (254-mm) JRCP placed over the 4-in (102-mm) PAGG, and includes a 41-ft (12.5-m) joint spacing with 1.25-in (38-mm) dowels. The shoulder is tied using 5/8-in (16-mm) hook bolts spaced 4.5 ft (1.4 m) apart, and the traffic lanes are tied using 24 in (610 mm) long, No. 5 (16-mm) bars at 41-in (1040-mm) centers. The steel design consists of W8.6 (nominal diameter of 0.331 in [8.4 mm]) wires at 6-in (152-mm) centers in the longitudinal direction, and W4 (nominal diameter of 0.225 in [5.7 mm] wires at 12-in (3-5-mm) centers in the transverse direction. The subgrade is an AASHTO A-2-4 material.

The performance of this section is shown in table 31. This pavement is still performing very well. While some transverse cracking has developed, it is not detracting from the performance of the pavement. The serviceability and roughness levels indicate a smooth-riding pavement surface.

Table 31. Design and outer lane performance data for MI 3, MI 5, and MI 6.

	MI 3		MI 5		MI 6
Pavement Type	JRCP 0.14% Steel		JRCP 0.14% Steel		JPCP
Year Built	1986		1984		1942
Thickness, in	10-in (254-mm)		10 in (254-mm)		10-in (254-mm)
Joint Spacing, ft	41 (12.5-m)		41 (12.5-m)		25 (7.6-m)
Dowel Diameter, in	1.25 (32-mm)		1.25 (32-mm)		None
Shoulder Type	Reinforced PCC Shoulder		Nonreinforced PCC Shoulder		PCC Shoulder
Base Type	4-in (102-mm) PAGG		4-in (102-mm) PAGG		5-in (127-mm) AGG
Subbase	10-in (2-54-mm) Sand		21-in (533-mm) Sand		None
Subgrade	AASHTO A-2-4		AASHTO A-2-4		AASHTO A-6
Performance Data	1987 Survey	1992 Survey	1987 Survey	1992 Survey	1992 Survey
Age at Survey, years	1	6	3	8	50
ESAL's, millions	2.8	11.5	3.1	8.5	34.1
Faulting, in	0.02	0.01	0.05	0.04	0.16
Det. Tr. Crks/mi	0	10	144	253	103
% Slabs Cracked	—	—	—	—	46
Long. Cracks, ft/mi	0	0	0	0	30
% Joints Spalled	0	0	0	0	8
PSR	4.8	3.8	4.2	3.4	—
IRI, in/mi	—	122	—	188	335

Michigan 4 (I-69, Charlotte)

This project, located on I-69 near Charlotte, was constructed in 1972 to evaluate the differences in performance of JRCP provided with different shoulder types. Although four total sections were constructed (one tied PCC shoulder, two experimental AC shoulders, and the standard AC shoulder), only two sections were selected for this study. Both sections are 9-in (229-mm) JRCP designs with transverse joints that are spaced at 71.2-ft (21.7-m) intervals and contain 1.25-in (32-mm) diameter dowels coated with liquid asphalt. These sections are constructed on a 4-in (102-mm) aggregate base over a 10-in (254-mm) sand subbase and an AASHTO A-4 subgrade. The PCC shoulder was tied to the mainline using 9/16-in (14-mm) hook bolts spaced 40 in (1020 mm) apart. The experimental design matrix and the performance summary data for the selected sections are shown in table 32.

Table 32. Experimental design matrix and performance summary for MI 4.

		PCC Shoulder		AC Shoulder	
		1987 Survey	1992 Survey	1987 Survey	1992 Survey
Age at Survey, years		16	21	16	21
ESAL's, millions		4.4	6.3	4.4	6.3
		4-1	4-1	4-2	4-2
9-in (229-mm) JRCP	Faulting, in	0.12	0.19	0.06	0.13
0.15% Steel	Det. Tr. Crks/mi	227	222	183	198
4-in (102-mm) AGG Base	Long. Cracks, ft/mi	40	129	143	143
10-in (254-mm) AGG Subbase	% Joints Spalled	0	0	6	6
71.2-ft (21.7-m) Joints	PSR	2.4	2.4	2.4	2.2
	IRI, in/mi	—	208	—	165

1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km

Performance

As seen in table 32, both sections in MI 4 are extensively cracked. Most of the high severity transverse cracks in MI 4-1 are faulted more than 0.3 in (7.6 mm) and many are also spalled. MI 4-2 is similarly distressed, but contains slightly fewer deteriorated cracks. The maximum faulting of high severity cracks in MI 4-2 was 0.59 in (15.0 mm).

The number of deteriorated transverse cracks on section 4-1 (PCC shoulder) has remained constant since the 1987 survey, while the number has increased on section 4-2. Joint spalling has not increased on either section since the 1987 survey.

The levels of faulting for both sections are approaching unacceptable values for long-jointed pavements. The section with the tied PCC shoulders is exhibiting a greater level of faulting than the section with AC shoulders, probably due to the large spacing of tie bars. Both sections exhibit the same increase in faulting (0.07 in [1.8 mm]) since the 1987 survey.

The roughness measurements for both sections are rather high, particularly for section 4-1. In addition, the serviceability ratings for both pavements indicate fairly rough riding pavements.

Lane-separation and lane-shoulder dropoffs are also prominent problems in MI 4. The lane separation in excess of 0.6 in (15 mm) and a lane-shoulder dropoff in excess of 0.8 in (20 mm) were recorded for these sections. This is even the case for the section with the tied PCC shoulder.

Michigan 5 (I-94, Paw Paw)

This single section was constructed in 1984 using recycled aggregate. Located on I-94 near Paw Paw, this section has a permeable aggregate base course and tied, nonreinforced concrete shoulders. The JRCP slab is 10 in (254 mm) thick with doweled transverse joints spaced at 41-ft (12.5-m) intervals. The subgrade on the project is an AASHTO A-2-4 material. This section is identical to MI 3, except MI 5 has tied, nonreinforced concrete shoulders while MI 3 contains tied, reinforced concrete shoulders using 5/8-in (16-mm) hook bolts at 4.5-ft (1.4-m) centers.

The performance of this section is shown in table 31. As shown in the table, MI 5 is extensively deteriorated. Almost 100 percent of the slabs in MI 5 contain two medium- to high-severity transverse cracks, each within 1 ft (0.3 m) of the third points between transverse joints. Faulting of the high-severity cracks is often in excess of 0.2 in (5 mm), and many of the short pavement sections (the sections delineated by the high severity cracks and joints) are visibly rocking under truck traffic. Additional low-severity cracks are developing at the approximate midpoints of the short pavement sections delineated by the cracks and joints.

The pavement has continued to deteriorate since its evaluation in 1987. In the intervening 5-year period, it has developed much more transverse cracking and has seen its serviceability drop off to 3.4. The roughness level measured on this pavement section is 188 in/mi (3.0 m/km), which is indicative of a moderately rough pavement.

As previously noted, in contrast to MI 5, MI 3 is performing very well. Very few deteriorated cracks are observed from MI 3, which was constructed and opened to traffic 2 years later than MI 5, but has now accumulated more traffic loadings than MI 5.

Michigan 6 (Davison Freeway, Detroit)

This single section is a 10-in (254-mm) JPCP located on the Davison Freeway near downtown Detroit. This section of Davison Freeway is a sunken urban highway (approximately 25 ft [7.6 m] below the ground level), with retaining walls and curbs on both sides of the roadway. It was constructed in 1942 and contains nondoweled joints spaced at 25-ft (7.6-m) intervals. The pavement rests on a 5-in (127-mm) granular base course placed on the clay subgrade. This project was included because it represents an older JPCP design that has been subjected to heavy traffic loadings.

Performance data for MI 6 are presented in table 31. Nearly 50 percent of the slabs in this section are cracked, and most of the cracks that are present are badly spalled and faulted. The average faulting for the section is 0.16 in (4.1 mm), with maximum values of 0.75 in (19 mm) recorded. This has undoubtedly contributed to the severe roughness of the pavement (335 in/mi [5.3 m/km]).

Missouri 1 (I-35, Bethany)

This project, constructed in 1977, consists of eight sections. Located in the northbound lanes of I-35, near Bethany, this project was constructed to evaluate the effect of coarse aggregate size and base type on pavement performance.(26)

Common to all sections are a 9-in (229-mm) JRCP and 1.25-in [32-mm] noncoated dowel bars at transverse joints spaced at 61.5-ft (18.7-m) intervals. The sections contain 0.11 percent longitudinal reinforcing steel, which consists of 0.28-in (7.1-mm) diameter bars at 6-in (150-mm) centers. The transverse steel consists of 0.22-in (5.6-mm) diameter bars at 12-in (305-mm) centers. The shoulder design throughout the project consists of 5-in (127-mm), permeable open-graded aggregate subbase and 8-in (203-mm) dense-graded aggregate surface. A 3-ft (0.9-m) wide segment of the shoulder adjacent to either side of the mainline pavement has a 2-in (51-mm) AC surface.

The experimental design matrix for this project is provided in table 33. Four base types (AGG, ATB, PATB, and CTB) and three maximum coarse aggregate sizes (2 in [51 mm], 1 in [25 mm], and 0.75 in [19 mm]) are included in the project, although a full factorial design is not available.

Traffic Loadings

Traffic data from 1992 indicates that this pavement section carries a two-way ADT of 8,600 vehicles per day, including 28 percent heavy trucks. Since its opening to traffic in 1977, it is estimated that this pavement has sustained 13.7 million 18-kip (80-kN) ESAL applications through 1992.

Table 33. Experimental design matrix for MO 1.

		4-in (102-mm) AGG	4-in (102-mm) ATB (5%, 60-70 pen)	4-in (102-mm) PATB (3%, 60-70 pen)	4-in (102-mm) CTB (4.5% cement)
9-in (229-mm) JRCP	2-in (51-mm) Max Size Aggregate[1]	MO 1-1			
0.1% Steel	1-in (25-mm) Max Size Aggregate	MO 1-4 MO 1-8[2]	MO 1-5	MO 1-6	MO 1-7
61.5-ft (18.7-m) Joints	0.75-in (19-mm) Max Size Aggregate	MO 1-2[2] MO 1-3			

[1] Burlington Limestone — not D-Cracking.
[2] Contains moisture barrier (4-mil [0.1-mm] thickness polyethylene).
MO 1-4 = SHRP 295000, MO 1-5 = SHRP 295058, MO 1-6 = SHRP 295081, MO 1-7 = SHRP 295091.

Performance

Table 34 summarizes the performance data for the outer lanes of the MO 1 sections. Figure 31 presents that performance data in graphical format. The distress data for MO 1 do not indicate any major differences in the performance of these sections. Considering the performance of the various base types, the levels of faulting, transverse cracking, and joint spalling do not appear to be significantly different. Pumping was observed on all sections constructed on aggregate bases, but not for those constructed on treated bases. However, the section constructed on PATB had fewer deteriorated transverse cracks. Nevertheless, differences in the overall performance of the various base types did not appear to be significant.

The effect of maximum coarse aggregate size can be assessed for those sections constructed on an aggregate base course. By comparing MO 1-1 (2 in [51 mm] maximum aggregate size) and MO 1-3 (0.75 in [19 mm] maximum aggregate size), the larger coarse aggregate is effective in preventing the deterioration of transverse cracks. However, the smaller coarse aggregate size is effective in reducing joint spalling.

Most of the longitudinal cracking observed on these sections is believed to be the result of high steel. The steel reinforcement was visible at many locations parallel to the longitudinal lane-lane joint, and longitudinal cracking emanated from these locations of high steel.

Roughness and Serviceability

Figure 32 illustrates the roughness measurements and serviceability ratings for the MO 1 sections. Whereas the roughness values indicate a significant difference in smoothness between the sections (with the smoothest riding sections being three of

Table 34. Summary of 1992 outer lane performance data for MO 1.

			4-in (102-mm) AGG		4-in (102-mm) ATB (5%, 60-70 pen)	4-in (102-mm) PATB (3%, 60-70 pen)	4-in (102-mm) CTB (4.5% cement)
			1992 Survey		1992 Survey	1992 Survey	1992 Survey
Age at Survey, years			15		15	15	15
ESAL's, millions			13.7		13.7	13.7	13.7
9-in (229-mm) JRCP 0.1% Steel 61.5-ft (18.7-m) Jts 1.25-in (32-mm) Dowels	2-in (51-mm) Max. Agg. Size		1-1				
		Faulting, in	0.09				
		Det. Tr. Crks/mi	0				
		Long. Cracks, ft/mi	286				
		% Joints Spalled	19				
		PSR	4.1				
		IRI, in/mi	117				
	1-in (25-mm) Max. Agg. Size		1-4	1-8*	1-5	1-6	1-7
		Faulting, in	0.06	0.06	0.05	0.06	0.06
		Det. Tr. Crks/mi	29	23	23	6	23
		Long. Cracks, ft/mi	0	367	0	372	269
		% Joints Spalled	13	6	6	19	25
		PSR	4.0	3.9	3.9	4.2	4.0
		IRI, in/mi	159	176	131	163	146
	0.75-in (19-mm) Max. Agg. Size		1-2*	1-3			
		Faulting, in	0.07	0.07			
		Det. Tr. Crks/mi	11	17			
		Long. Cracks, ft/mi	69	0			
		% Joints Spalled	12	6			
		PSR	4.0	3.9			
		IRI, in/mi	118	126			

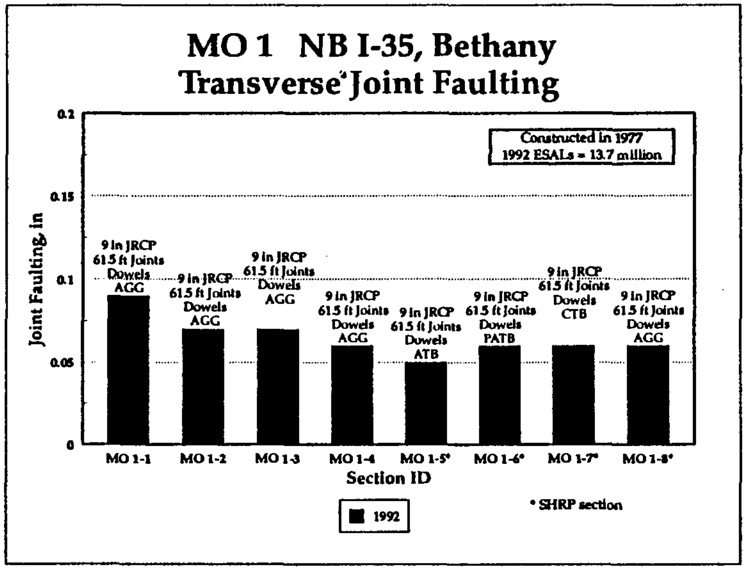
MO 1-1 and 1-3 have A-7-6 subgrade. 1 in = 25.4 mm
MO 1-2 has A-6 subgrade. 1 ft = 0.305 m
Other sections have A-7 subgrade 1 mi = 1.61 km
* Contains moisture barrier (4-mil [0.1-mm] thickness polyethylene).
MO 1-4 = SHRP 295000, MO 1-5 = SHRP 295058, MO 1-6 = SHRP 295081, MO 1-7 = SHRP 295091.

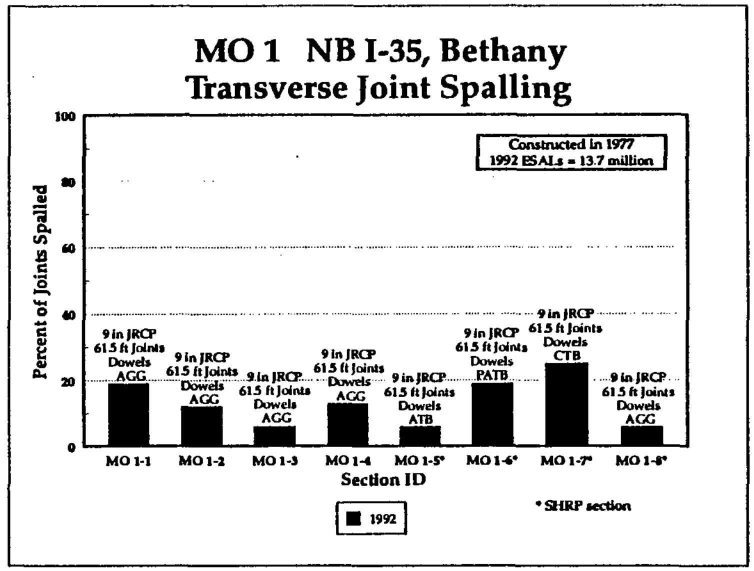
the five sections constructed on aggregate base), no apparent trends in the roughness measurements were recorded. No significant differences in rideability are evident from the serviceability ratings.

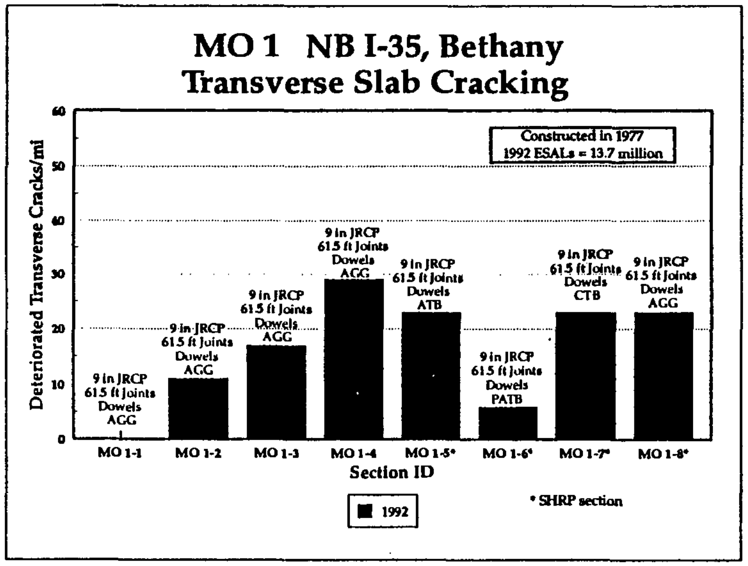
Summary

The performance of the MO 1 experimental sections is summarized below:

- No significant difference is observed between the performance of the pavement sections constructed on various base types. Levels of faulting, spalling, and transverse cracking are comparable for all sections, although pumping was







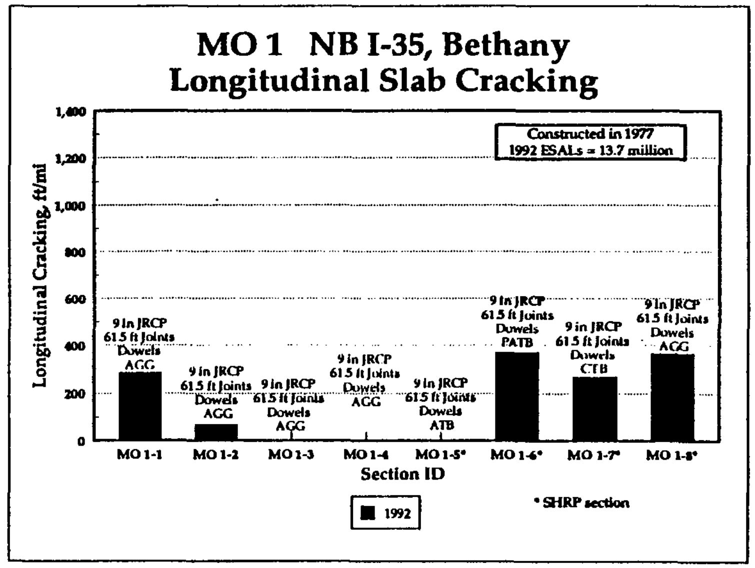
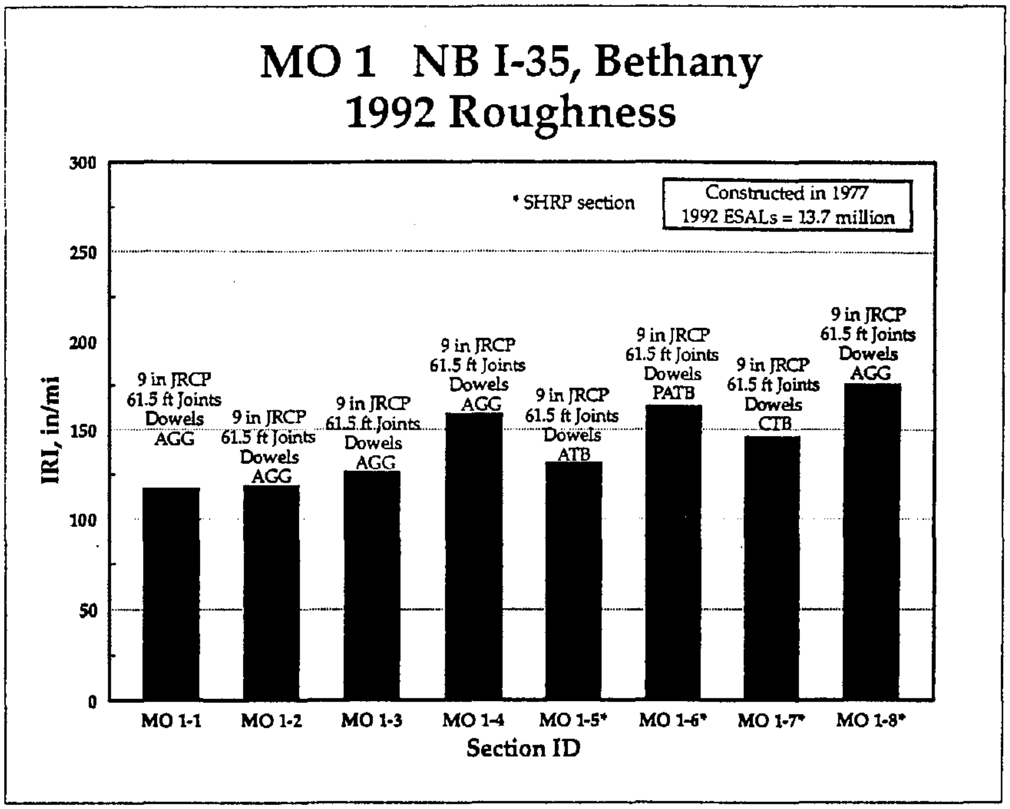


Figure 31. Performance summary for MO 1.



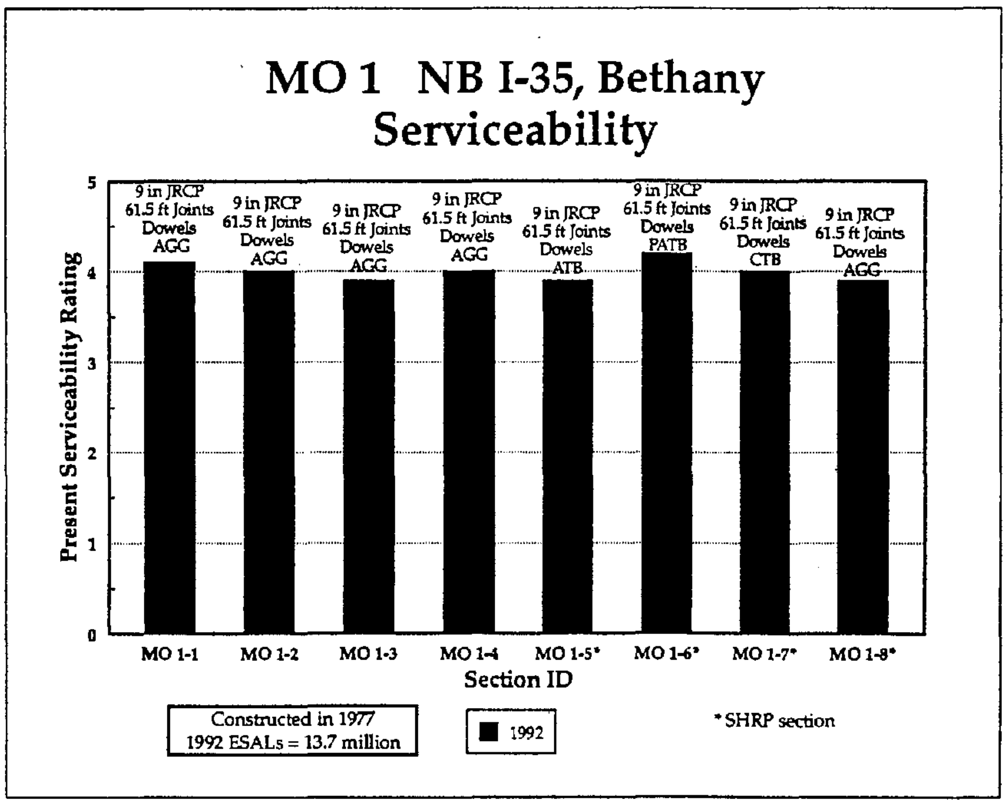


Figure 32. Roughness and serviceability for MO 1.

only observed on the AGG sections. The section constructed on PATB did exhibit less transverse cracking than the other sections, suggesting that a reduction in transverse crack deterioration is related to the removal of excess water from the pavement structure.

- The section with the 2-in (51-mm) size coarse aggregate (MO 1-1) shows less crack deterioration than the sections constructed with the 0.75-in (19-mm) size coarse aggregate (MO 1-2 and MO 1-3). The larger size aggregate is expected to provide better load transfer at transverse cracks, reducing their rate of deterioration. The sections with the smaller coarse aggregate did exhibit less joint spalling, however.

- No apparent trends exist regarding the roughness levels of the various sections.

- The longitudinal cracking that developed on the project is believed to be the result of high reinforcing steel.

New Jersey 2 (RT 130, Yardville)

A part of Route 130 near Yardville, New Jersey is the one of the oldest sections included in this study. Once a major access route to New York City, it was constructed in 1951 and is typical of New Jersey's current standard concrete pavement design. The pavement is a 10-in (254-mm) JRCP that rests on an aggregate base and subbase material. The slabs are 78.5-ft (23.9-m) long and are constructed with expansion joints at that interval. The longitudinal steel consists of 3/8-in (9.5-mm) bars at 7.5-in (190-mm) centers. Load transfer is provided by stainless steel-wrapped dowel bars, 1.25-in (32-mm) in diameter.

The performance of this section is shown in table 35. This section continues to exhibit good performance, as the levels of faulting, spalling, and cracking are all at acceptable values. Overall, the pavement is in good condition for being so old and having carried so many ESAL applications. This performance is attributed, in part, to the high quality of the load transfer device.

However, the PSR has dropped considerably since 1987, and the roughness level is approaching a significant level. This result may indicate that the rate of deterioration for this 41-year-old pavement has increased.

New Jersey 3 (I-676, Camden)

This experimental project is located on I-676 near Camden. Built in 1979, the project is a drainage study evaluating the performance of pavement sections with open-graded aggregate bases (PAGG) and bituminous-stabilized open-graded base layers (PATB). Both sections included in this project have 9-in (229-mm) JRCP slabs, 78.5-ft (23.9-m) transverse joint spacings (every joint is an expansion joint), and

Table 35. Design and outer lane performance data for NJ 2.

	NJ 2	
Pavement Type	JRCP 0.14% Steel	
Year Built	1951	
Thickness, in	10	
Joint Spacing, ft	78.5	
Dowel Diameter, in	1.25	
Base Type	5-in (127-mm) AGG	
Subbase	7-in (178-mm) AGG	
Subgrade	AASHTO A-4	
Performance Data	1987 Survey	1992 Survey
Age at Survey, years	36	41
ESAL's, millions	34.8	38.2
Faulting, in	0.06	0.07
Deteriorated Tr. Crks/mi	24	19
Long. Cracks, ft/mi	10	10
% Joints Spalled	20	20
PSR	3.8	3.1
IRI, in/mi	—	192

1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km

1.25-in (32-mm) diameter stainless steel-wrapped dowel bars. A filter fabric is placed full-width beneath both of the open-graded layers and above the lime-flyash stabilized subgrade. The longitudinal steel design consists of 3/8-in (9.5-mm) bars at 7.5-in (190-mm) centers. The simplified design matrix is presented in table 36.

Performance

The performance data for the outer traffic lane are provided in table 37. Both of these sections are performing well after 13 years of service. No appreciable difference in any of the performance indicators is noticeable between the sections with the different base designs. No appreciable changes have occurred since 1987.

The levels of roughness are somewhat high, which is probably due to the wide expansion joints (0.75 in [19 mm]) present at 78.5-ft (23.9-m) intervals. Similarly, the joint spalling exhibited by each section is probably due to the construction of the expansion joints.

Table 36. Experimental design matrix for NJ 3.

	4-in (102-mm) PAGG	4-in (102-mm) PATB (2.5%, AC-20)
9-in (229-mm) JRCP 0.16% Steel 78.5-ft (23.9-m) Exp. Joints	NJ 3-1	NJ 3-2

Table 37. Summary of 1987 and 1992 outer lane performance data for NJ 3.

		4-in (102-mm) PAGG A-1-a Subgrade		4-in (102-mm) PATB (2.5%, AC-20) A-2-4 Subgrade	
		1987 Survey	1992 Survey	1987 Survey	1992 Survey
Age at Survey, years		8	13	8	13
ESAL's, millions		4.9	12.6	4.9	12.6
		NJ 3-1	NJ 3-1	NJ 3-2	NJ 3-2
9-in (229-mm) JRCP	Faulting, in	0.05	0.04	0.06	0.03
0.16% Steel	Det. Tr. Crks/mi	0	0	0	0
	Long. Cracks, ft/mi	0	0	0	0
78.5-ft (23.9-m)	% Joints Spalled	0	29	43	43
Expansion Joints	PSR	3.6	—	3.5	—
	IRI, in/mi	—	191	—	199

All section have 4-in LFAS subbase and AC shoulders.

1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km

New York 1 (RT 23, Catskill)

Route 23 between Catskill and Cairo, New York, is the site of an experimental project constructed in 1968. The purpose of this project (which consists of 30 sections representing 8 different designs) is to evaluate the effects of load transfer, joint orientation, base type, joint spacing, and pavement type on pavement performance.[27,28]

The designs included in this project are shown in table 38. All slabs are 9 in (229 mm) thick and the subgrade for the project varies from an AASHTO A-1-a to A-2-4 material. Load transfer, where applicable, is provided by ACME two-part malleable iron devices. AC shoulders are provided for each section.

Traffic Loadings

This four-lane divided roadway is currently subjected to a two-way ADT of 9,900 vehicles per day, including 9 percent heavy trucks. In 1987, it had sustained

Table 38. Experimental design matrix for NY 1.

		Perpendicular Joints		Skewed Joints
		Load Transfer	No Load Transfer	No Load Transfer
9-in (229-mm) JPCP **20-ft (6.1-m) Joints**	4-in (102-mm) AGG	NY 1-6*		
	3-in (76-mm) ATB (2.5-4%, 60-70 pen)	NY 1-1*	NY 1-7 NY 1-8a*	NY 1-8b*
	4-in (102-mm) Soil Cement (8-10% cement)		NY 1-5a	NY 1-5b
9-in (229-mm) JRCP **0.20% Steel** **61-ft (18.6-m) Joints**	4-in (102-mm) AGG	NY 1-4*		
	3-in (76-mm) ATB (2.5-4%, 60-70 pen)	NY 1-2 NY 1-3*		

* 1987 Section
All sections contain an 8-in (203-mm) aggregate subbase and AC shoulders.

approximately 3.1 million 18-kip (80 kN) ESAL applications in the outer traffic lane, and through 1992 had received approximately 5.5 million ESAL applications.

Performance

The performance data for the NY 1 experimental sections are summarized in table 39. The key distress data are illustrated in figure 33.

All sections are in remarkably good condition after 24 years of service. In terms of faulting and joint spalling, the short-jointed JPCP sections are performing better than the long-jointed JRCP sections. The average joint faulting of the 61-ft (18.6-m) jointed sections is considerably greater than that of the 20-ft (6.1-m) jointed sections. Joint spalling is also more severe in the long-jointed sections.

The effect of base type on pavement performance is not clear. Among the long-jointed sections, the section constructed on ATB is more severely spalled than the one constructed on aggregate base. However, the ATB section does not exhibit any deteriorated transverse cracks, and the faulting levels are about the same for each section.

For the short-jointed sections, no significant differences could be observed in the performance of the sections on different base types. None of the short-jointed sections is significantly faulted or spalled, although the section constructed on an aggregate base does exhibit more transverse slab cracking than the rest. This result is somewhat surprising, as more cracking is expected to occur on the sections with treated bases due to increased thermal curling stresses.

Table 39. Summary of 1987 and 1992 outer lane performance data for NY 1.

			Perpendicular Joints				Skewed Joints	
			Load Transfer		No Load Transfer		No Load Transfer	
			1987	1992	1987	1992	1987	1992
Age at Survey, years			19	24	19	24	19	24
ESAL's, millions			3.1	5.5	3.1	5.5	3.1	5.5
9-in (229-mm) JPCP 20-ft (6.1-m) Joints	4-in (102-mm) AGG		1-6	1-6				
		Faulting, in	0.03	0.02				
		Det. Tr. Crks/mi	35	44				
		% Slabs Cracked	10	17				
		Long. Cracks, ft/mi	264	475				
		% Joints Spalled	13	13				
		PSR	3.9	3.8				
		IRI, in/mi	—	118				
	3-in (76-mm) ATB (2.5-4%, 60-70 pen)		1-1	1-1	1-8a	1-8a	1-8b	1-8b
		Faulting, in	0.02	0.02	0.01	0.01	0.03	0.03
		Det. Tr. Crks/mi	0	18	9	26	0	18
		% Slabs Cracked	0	7	3	10	0	7
		Long. Cracks, ft/mi	0	70	290	290	0	0
		% Joints Spalled	6	6	0	0	0	0
		PSR	4.0	3.7	4.1	4.2	3.8	3.9
		IRI, in/mi	—	106	—	112	—	111
	4-in (102-mm) Soil Cement (8-10% cement)					1-5a		1-5b
		Faulting, in				—		—
		Det. Tr. Crks/mi				20		51
		% Slabs Cracked				11		16
		Long. Cracks, ft/mi				0		0
		% Joints Spalled				0		3
		PSR				—		—
		IRI, in/mi				—		—
9-in (229-mm) JRCP 0.20% Steel 61-ft (18.6-m) Joints	4-in (102-mm) AGG		1-4	1-4				
		Faulting, in	0.09	0.14				
		Det. Tr. Crks/mi	0	9				
		Long. Cracks, ft/mi	0	138				
		% Joints Spalled	9	9				
		PSR	3.4	3.4				
		IRI, in/mi	—	177				
	3-in (76-mm) ATB (2.5-4%, 60-70 pen)		1-3	1-3				
		Faulting, in	0.14	0.16				
		Det. Tr. Crks/mi	0	0				
		Long. Cracks, ft/mi	0	35				
		% Joints Spalled	64	64				
		PSR	3.6	3.1				
		IRI, in/mi	—	117				

All sections contain an 8-in aggregate subbase and AC shoulders.

1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km

The effect of the ACME load transfer devices in reducing faulting can be observed by comparing the performance of NY 1-1 and 1-8a. The distress data indicate no significant difference in the performance of these sections, suggesting that the light traffic to which this project is exposed may not be a true test of the load transfer devices. Other studies conducted in New York have indicated that these devices corroded and could not be relied upon to provide positive, long-term load transfer, often failing within 5 years.[29]

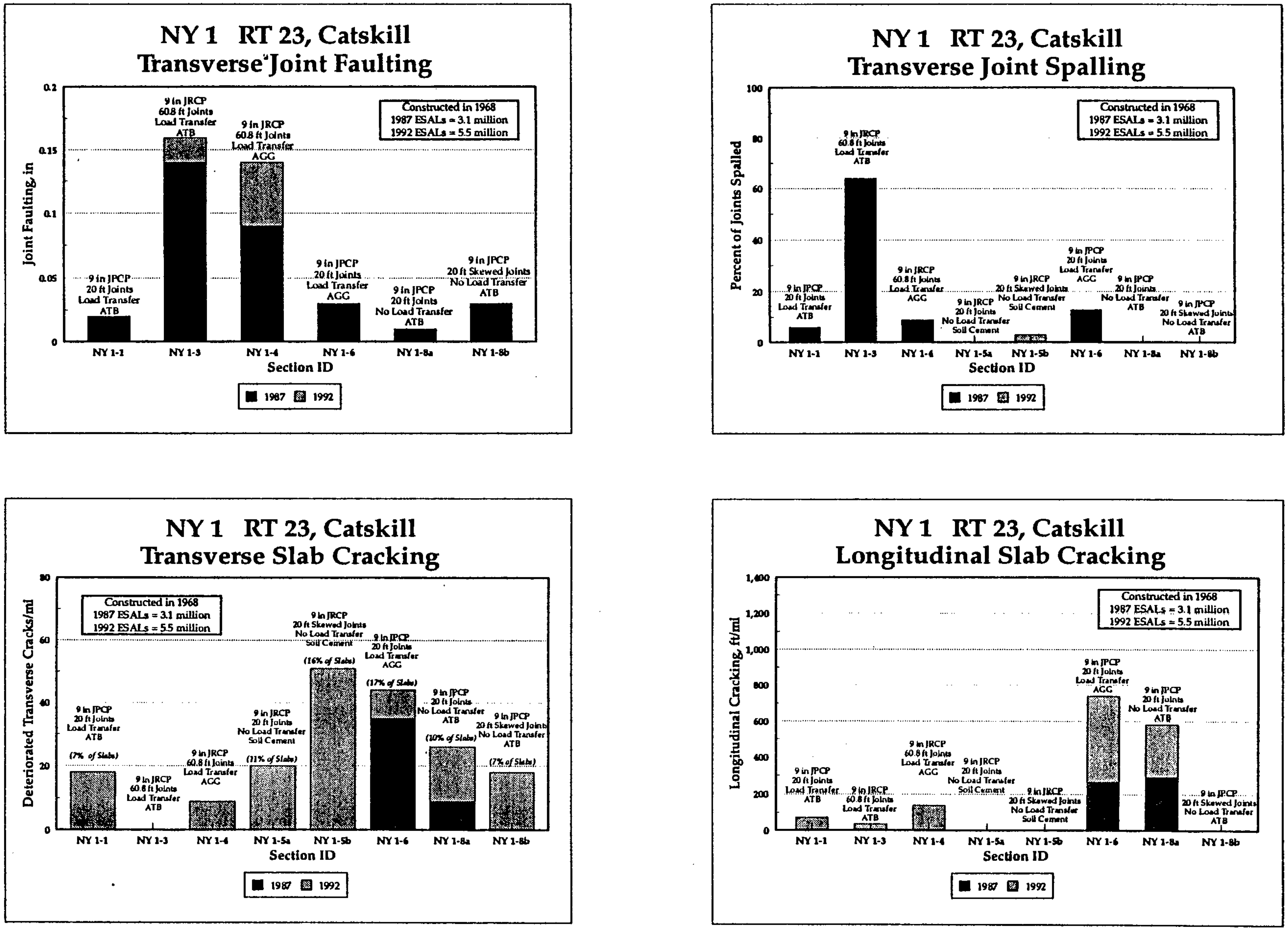


Figure 33. Performance summary for NY 1.

No definitive conclusions can be drawn regarding the effectiveness of skewed transverse joints. The section with the skewed joints displays more faulting than the corresponding section with perpendicular joints, although this difference is not significant. The section with perpendicular joints, however, displays more transverse slab cracking than the section with skewed joints.

A fair amount of longitudinal cracking was observed on several sections, probably related to construction practices. For example, the longitudinal cracking on NY 1-6 occurred where the traffic lanes are widely separated (ranging from 0.8 to 3 in [20 to 76 mm]) along the centerline joint (tie bars at 30-in [762-mm] centers). The wide lane separation can cause additional transverse cracking that is unrelated to the design features being evaluated.

Roughness and Serviceability

Roughness measurements and serviceability ratings for each section are illustrated in figure 34. The roughest section is NY 1-4, a long-jointed section on an aggregate base. The roughness levels for the other sections are all comparable and are representative of relatively smooth-riding pavements.

The pavements with the lowest serviceability ratings are the long-jointed JRCP designs. These sections are approaching critical serviceability values. The other sections are providing good rideability, with no significant changes in serviceability since 1987.

5-Year Performance Trends

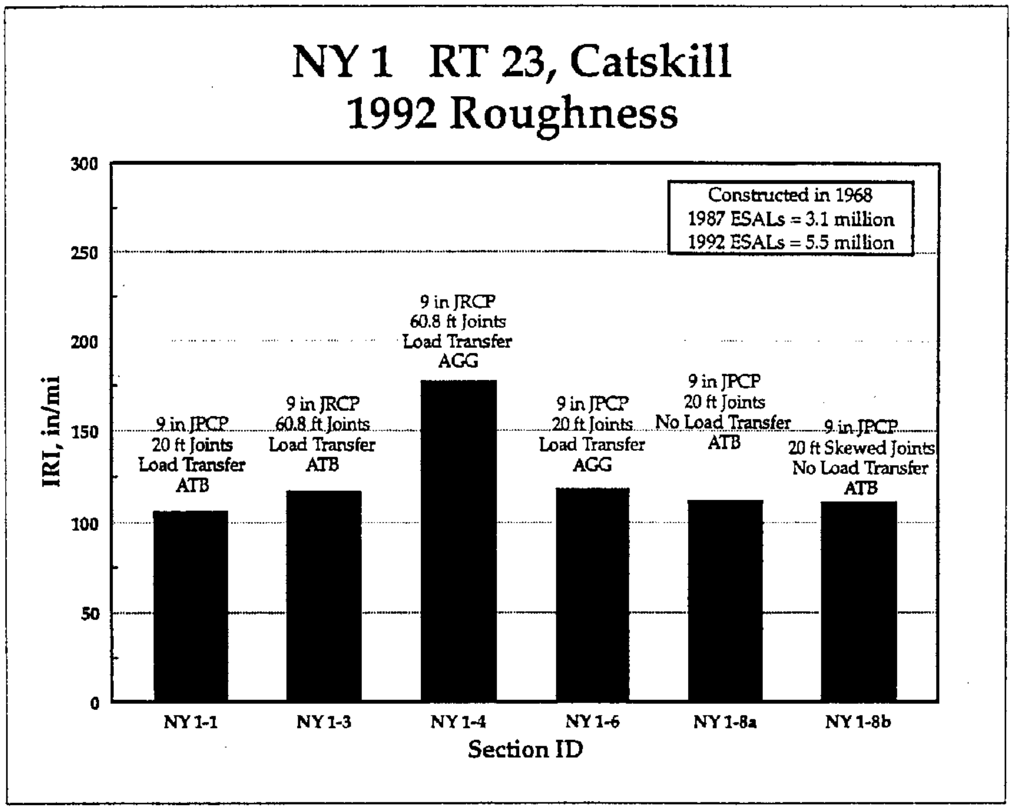
The 5-year performance trends for these sections are shown in figure 35. No change in joint faulting or spalling has occurred for any of the sections. However, the number of deteriorated transverse cracks has increased for nearly every section, and the amount of longitudinal cracking has also increased slightly for a few sections.

Summary

The performance of the NY 1 sections is summarized below:

- The short-jointed JPCP sections are performing better than the long-jointed JRCP sections. The long-jointed JRCP sections are exhibiting substantially more faulting and spalling than the short-jointed sections, and overall are rougher.

- The effect of base type on pavement performance is not clear. No difference in performance is observed between the short-jointed sections constructed on different base types. However, the long-jointed JRCP section constructed on ATB has considerably more spalling than the long-jointed JRCP section constructed on AGG; the ATB section, on the other hand, is much smoother than the AGG section.



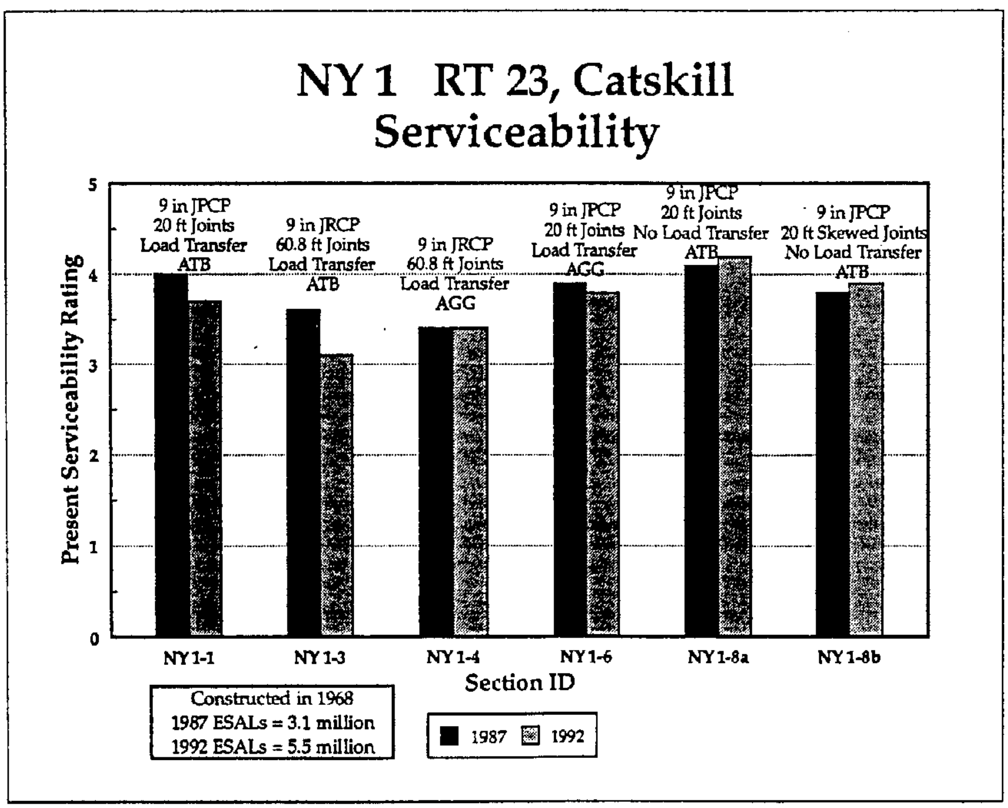


Figure 34. Roughness and serviceability for NY 1.

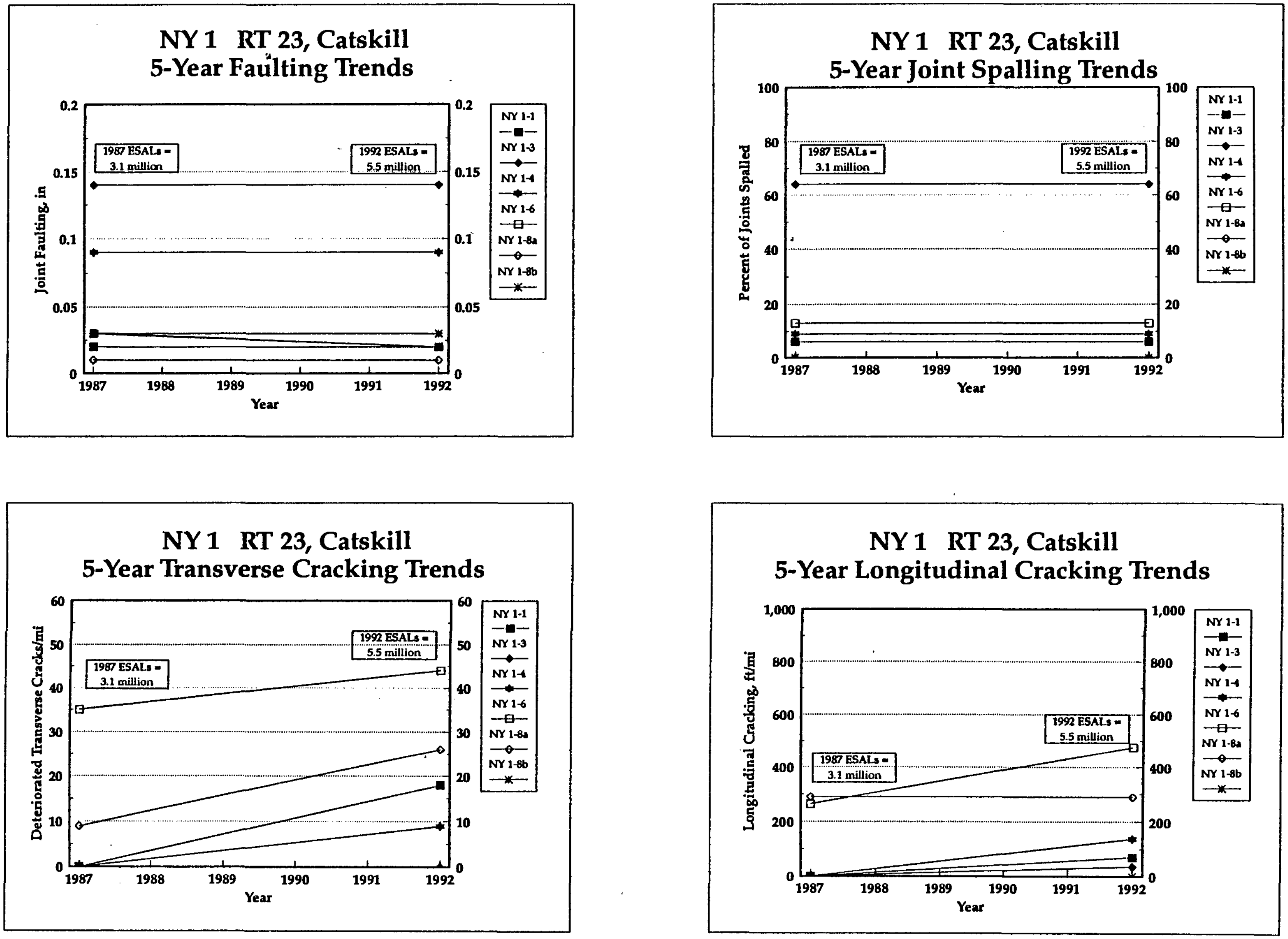


Figure 35. 5-year performance trends for NY 1.

- At this time, no performance benefits are apparent for the sections constructed with the ACME load transfer device. The faulting levels for sections with and without the load transfer device are about the same. This may be partially attributed to the low level of traffic present on the pavement.

- No definitive conclusions could be drawn regarding the effectiveness of joint skewing. The skewed section had slightly more faulting than its nonskewed counterpart, although it also had fewer deteriorated cracks.

- With the exception of the long-jointed JRCP, the pavement sections are providing satisfactory rideability. The long-jointed pavements are the roughest sections, probably due to the severe faulting that has developed.

- With the exception of transverse cracking, no significant changes in pavement condition have occurred for these sections over the last 5 years.

New York 2 (I-88, Otego)

In 1975, an experimental project was constructed on I-88 near Otego, New York. The purpose of the study was to evaluate the performance of jointed plain concrete pavements (which had fared well on NY 1) under Interstate traffic loading conditions.[30] Design variables include pavement type, joint spacing, and shoulder type. In addition, the effect of sealing the lane-shoulder joint was investigated. Although a total of 15 different designs were constructed (replicated in each direction), only 4 were selected for the original study.

All slabs are 9 in (229 mm) thick and contain epoxy-coated I-beams for load transfer. Aggregate bases are common to all sections, as is the AASHTO A-1-a subgrade. Three sections contain a tied PCC shoulders, with tie bars spaced 40 in (1020 mm) apart. The experimental layout of this project is shown in table 40.

Traffic Loadings

Since its opening to traffic in 1975 and through 1992, this four-lane divided roadway has accumulated 5.8 million 18-kip (80-kN) ESAL applications in the outer lane, an increase from 1.6 million ESAL applications accumulated through 1987. The pavement is currently subjected to a two-way ADT of 9,100 vehicles per day, including 19 percent heavy trucks.

Performance

The performance data for NY 2 are summarized in table 41 and illustrated in figure 36. This information shows that all of the NY 2 sections are in excellent condition. Faulting is practically nonexistent on these sections, and joint spalling is also minimal.

Table 40. Experimental design matrix for NY 2.

	PCC Shoulder	AC Shoulder
9-in (229-mm) JPCP 20-ft (6.1-m) Joints Sealed Lane-Shlder Jt	NY 2-3	
9-in (229-mm) JPCP 20-ft (6.1-m) Joints Nonsealed Lane-Shlder Jt	NY 2-9	
9-in (229-mm) JPCP 26.7-ft (8.1-m) Joints	NY 2-11	
9-in (229-mm) JRCP 0.2% Steel 63.5-ft (19.4-m) Joints		NY 2-15

NY 2-3 & 2-15 have 4-in (102-mm) aggregate base and 8-in (203-mm) aggregate subbase.
NY 2-9 & 2-11 have 6-in (152-mm) aggregate base and no subbase.

The only distresses that are present in significant amounts are transverse and longitudinal cracking. All JPCP sections exhibit some deteriorated transverse cracks, ranging from 9 to 21 percent of the slabs. The longer-jointed JPCP section shows the same amount of transverse cracking as one of the shorter-jointed JPCP sections, although the underlying support is not the same for each section.

Sections 2-11 and 2-15 both display longitudinal cracking. All of the longitudinal cracking in section 2-11 occurred in the driving lane, mostly between the middle of the slab and the lane-shoulder joint. In the adjacent areas, the PCC shoulder is also cracked longitudinally through a considerable length of the section. The New York DOT believes that this longitudinal cracking is the result of differential frost heave.[30]

The lane-shoulder joint on NY 2-9 was left unsealed as a part of the experiment. Consequently, this joint is open to infiltration of water and incompressibles that can cause a number of distresses at this location, including pumping (which, in turn, can cause cracking because of loss of support) and spalling. Although some spalling did occur along the lane-shoulder joint in this section, very little transverse cracking occurred.

No major differences in the performance of the different pavement types can be made at this time. The levels of faulting and spalling are about the same, and both are providing good rideability. However, the JPCP sections have developed some transverse cracking, whereas the JRCP section has not.

Roughness and Serviceability

Roughness measurements and serviceability ratings for the NY 2 sections are illustrated in figure 37. This figure shows that the NY 2 sections are extremely

Table 41. Summary of 1987 and 1992 outer lane performance data for NY 2.

		PCC Shoulder		AC Shoulder	
		1987 Survey	1992 Survey	1987 Survey	1992 Survey
Age at Survey, years		12	17	12	17
ESAL's, millions		1.6	5.8	1.6	5.8
9-in (229-mm) JPCP		2-3	2-3		
20-ft (6.1-m) Joints	Faulting, in	0.01	0.01		
1-in (25-mm) I-Beam	Det. Tr. Crks/mi	35	55		
4-in (102-mm) AGG Base	% Slabs Cracked	13	21		
Sealed Lane-Shlder Joint	Long. Cracks, ft/mi	0	0		
	% Joints Spalled	0	0		
	PSR	4.2	—		
	IRI, in/mi	—	108		
9-in (229-mm) JPCP		2-9	2-9		
20-ft (6.1-m) Joints	Faulting, in	0.02	0.01		
1-in (25-mm) I-Beam	Det. Tr. Crks/mi	25	25		
6-in (152-mm) AGG Base	% Slabs Cracked	9	9		
Nonsealed Lane-Shlder	Long. Cracks, ft/mi	543	623		
Joint	% Joints Spalled	0	0		
	PSR	4.0	3.9		
	IRI, in/mi	—	91		
9-in (229-mm) JPCP		2-11	2-11		
26.7-ft (8.1-m) Joints	Faulting, in	0.01	0.01		
1-in (25-mm) I-Beam	Det. Tr. Crks/mi	26	40		
6-in (152-mm) AGG Base	% Slabs Cracked	13	20		
	Long. Cracks, ft/mi	0	0		
	% Joints Spalled	0	0		
	PSR	4.1	4.1		
	IRI, in/mi	—	98		
9-in (229-mm) JRCP				2-15	2-15
63.5-ft (19.4-m) Joints	Faulting, in			0.02	0.02
1-in (25-mm) I-Beam	Det. Tr. Crks/mi			0	0
4-in (102-mm) AGG Base	Long. Cracks, ft/mi			234	284
	% Joints Spalled			0	0
	PSR			4.0	4.3
	IRI, in/mi			—	89

NY 2-3 & 2-15 have and 8-in (203-mm) aggregate subbase.
NY 2-9 & 2-11 have no subbase.
All sections constructed on an A-1-a subgrade.

1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km

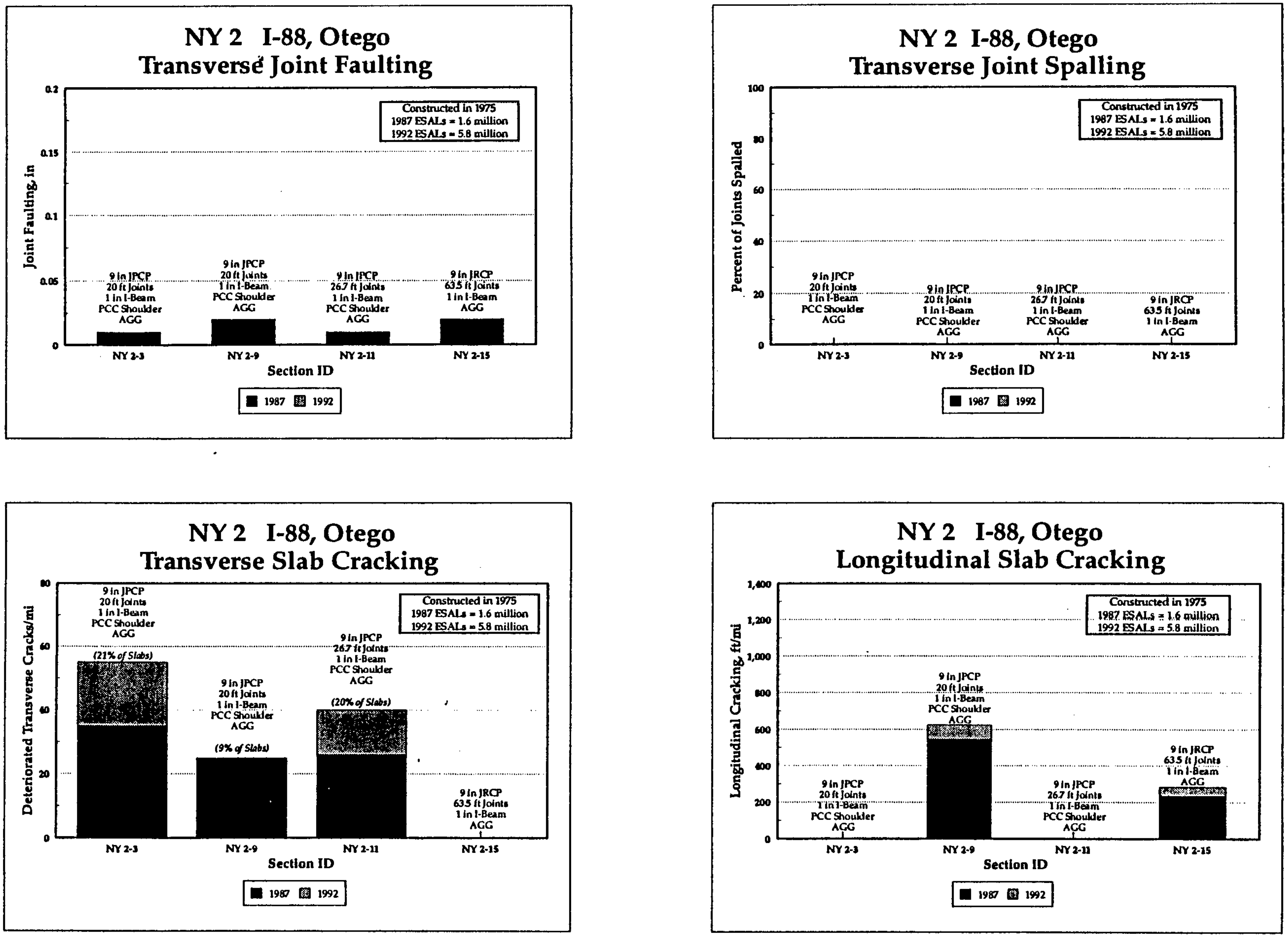
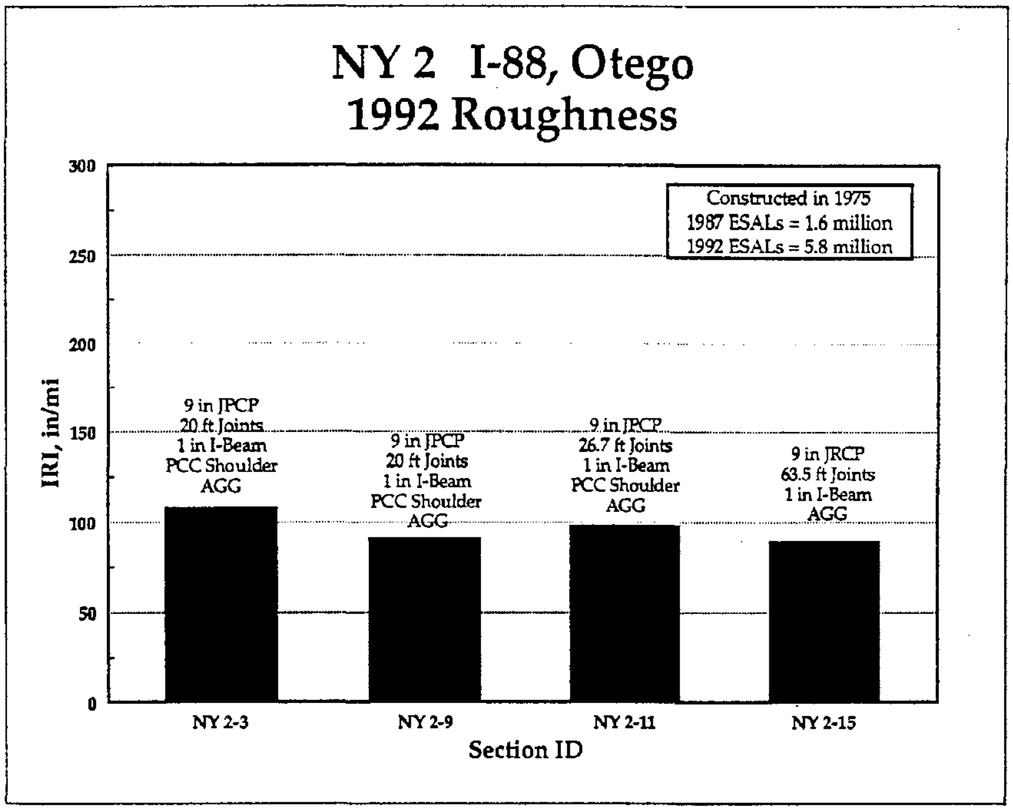


Figure 36. Performance summary for NY 2.



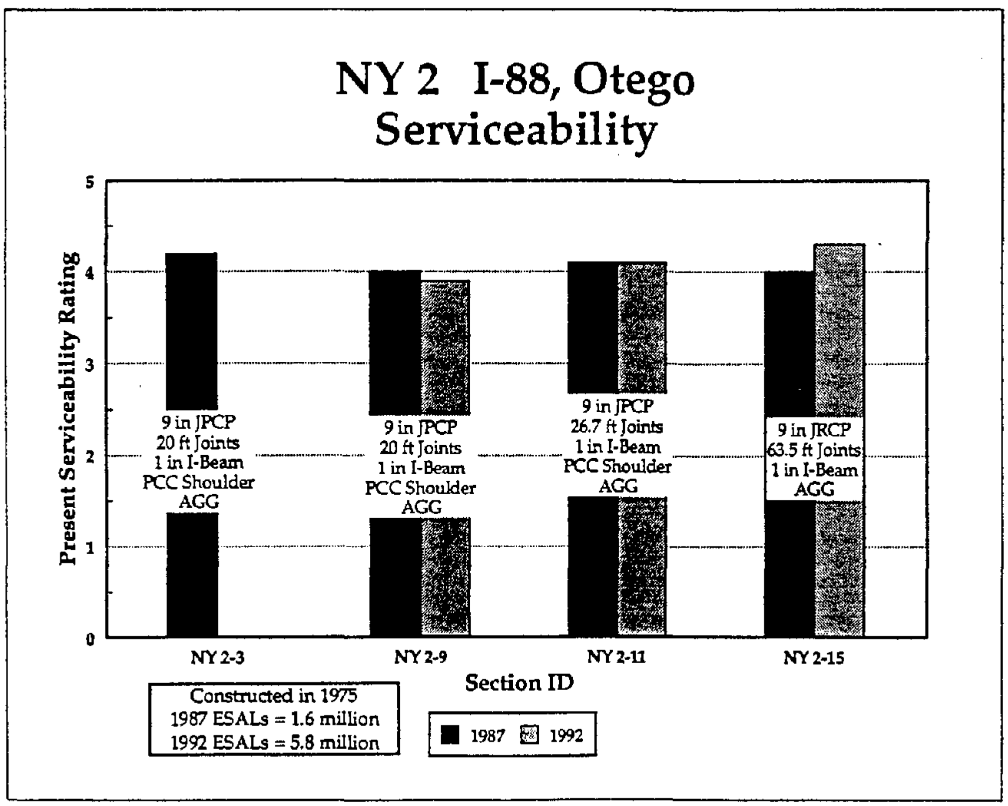


Figure 37. Roughness and serviceability for NY 2.

smooth, typically showing roughness measurements less than 100 in/mi (1.6 m/km) and PSR values near 4.0. Since 1987, no significant changes have occurred in the serviceability of the pavement.

5-Year Performance Trends

Figure 38 illustrates the 5-year performance trends for the NY 2 sections. Virtually no changes have occurred in the amount of joint faulting or spalling, although some increase in transverse cracking has occurred for sections 2-3 and 2-11. In addition, sections 2-9 and 2-15 are showing a small increase in the amount of longitudinal cracking.

Summary

The performance of the NY 2 sections may be summarized as follows:

- All sections are in relatively good condition, with very little joint faulting or joint spalling.

- All JPCP sections exhibit some deteriorated transverse cracks, whereas the JRCP sections have none. The JPCP section with longer joint spacing (26.7 ft [8.1 m]) showed the same amount of transverse cracking as one of the 20-ft (6.1-m) JPCP sections. This result may suggest that even the 20-ft (6.1-m) slabs are too long to prevent slab cracking, and the movement to a slightly longer joint spacing is not enough for additional cracking to occur.

- Two sections are exhibiting longitudinal cracking, which is attributed to differential frost heave of the subgrade.

- One section whose longitudinal lane-shoulder joint was left unsealed is not exhibiting performance different from other sections. While some spalling has occurred along the lane-shoulder joint, very little transverse cracking has developed.

- For all sections, roughness and serviceability levels are indicative of a smooth-riding pavement.

- Since the 1987 survey, no significant changes in pavement distress or serviceability have occurred.

<u>Ohio 1 (U.S. 23, Chillicothe)</u>

This experimental project, located on U.S. 23 near Chillicothe, was constructed in 1973 to evaluate the effect of different design variables on pavement performance.[31] The experimental variables for this project include base type, joint spacing, pavement type, and dowel coatings, as shown in table 42. All slabs are 9 in (229 mm) thick and the subgrade ranges from an AASHTO A-4 to an AASHTO A-6 material.

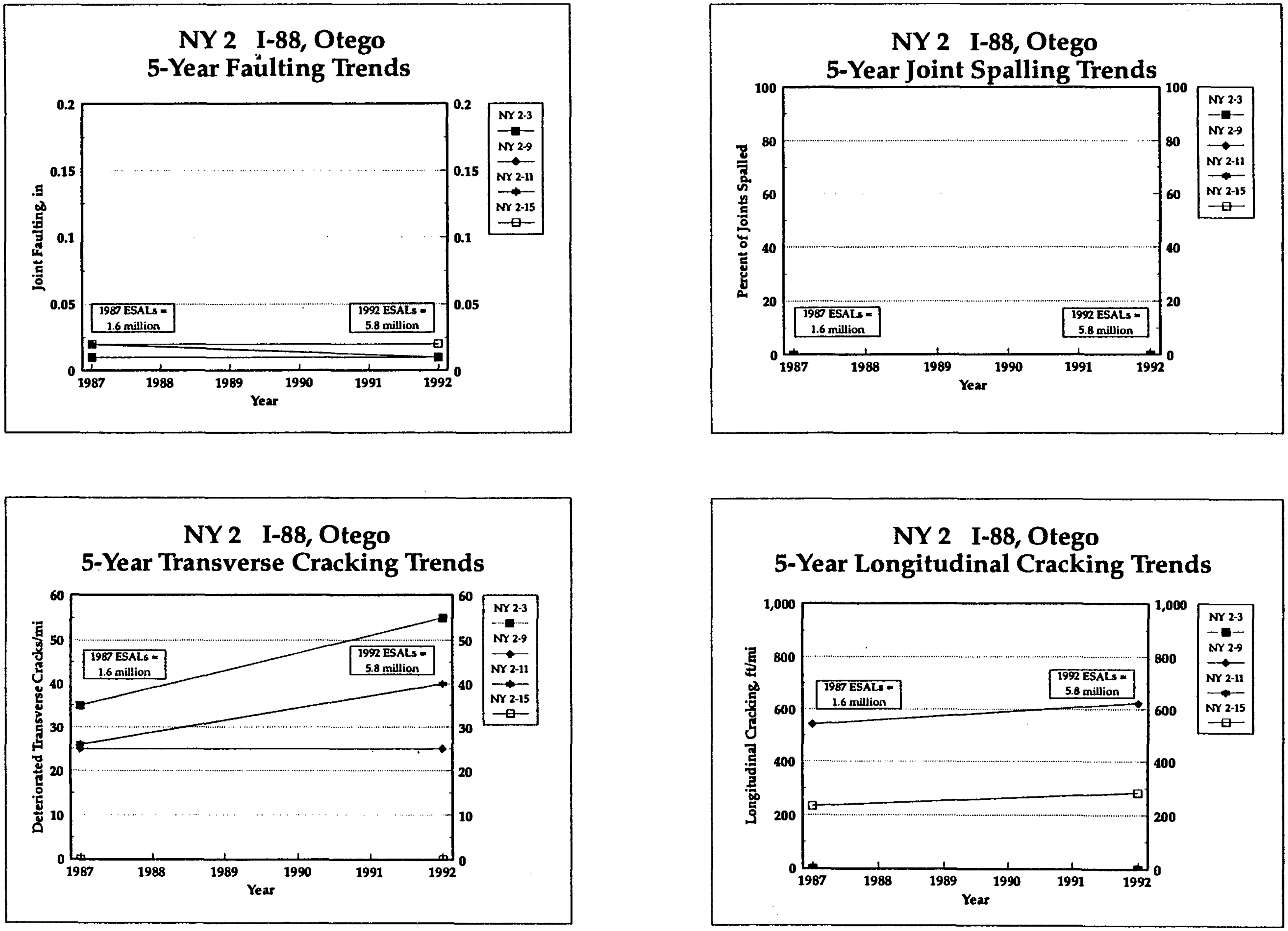


Figure 38. 5-year performance trends for NY 2.

Table 42. Experimental design matrix for OH 1.

		7.5-in (190-mm) Aggregate Base	**4-in (102-mm) ATB (5.7%, AC-20)**
9-in (229-mm) JRCP 0.09% Steel 21-ft (6.4-m) Jts	1.25-in (32-mm) Dowels (paint/grease)	OH 1-10*	OH 1-3*
	1.25-in (32-mm) Plastic-Coated Dowels	OH 1-6*	
9-in (229-mm) JRCP 0.09% Steel 40-ft (12.2-m) Jts	1.25-in (32-mm) Dowels (paint/grease)	OH 1-1* OH 1-2 OH 1-8 OH 1-9*	OH 1-4*
	1.25-in (32-mm) Plastic-Coated Dowels	OH 1-7*	
9-in (229-mm) JPCP 17-ft (5.2-m) Skewed Jts	No Dowels		OH 1-5

* 1987 section
No section contains a subbase course.

Traffic Loadings

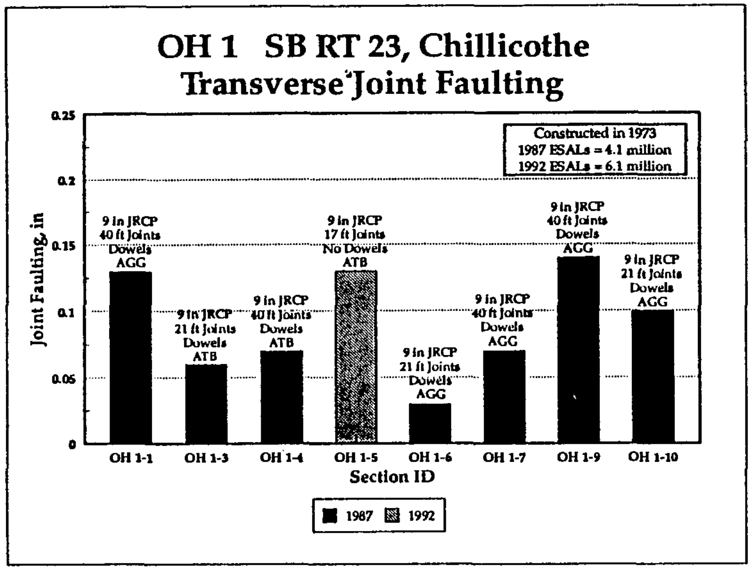
Through 1992, this pavement, a four-lane divided roadway, has sustained 6.1 million 18-kip (80 kN) ESAL applications in the outer lane, as compared to a total of 4.1 million ESAL applications in 1987. The current two-way ADT for this roadway is 17,000 vehicles per day, which includes 11 percent heavy trucks.

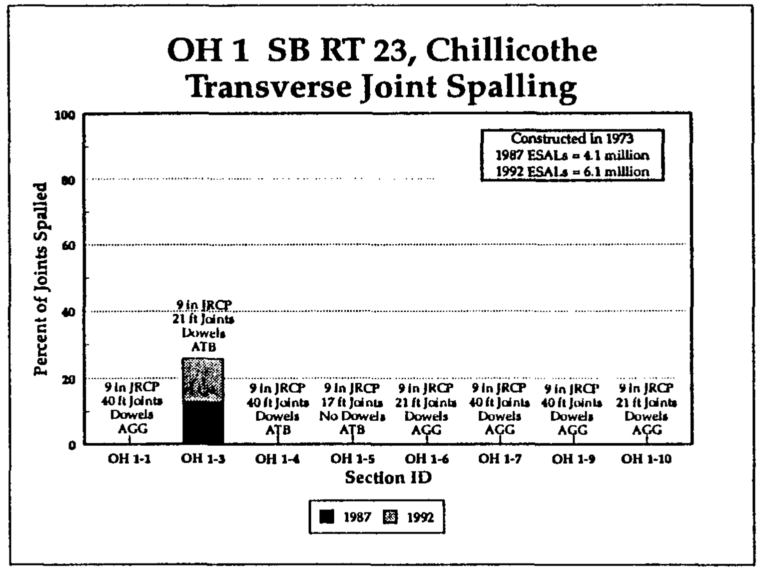
Performance

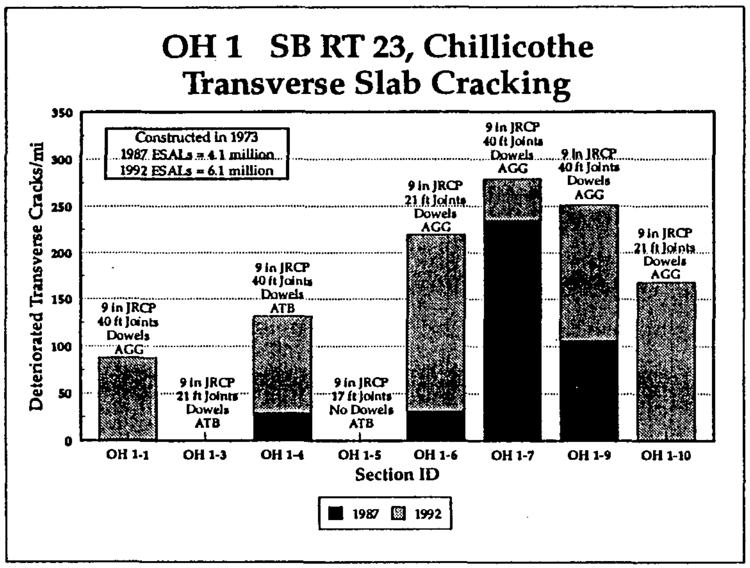
The performance data for OH 1 are summarized in table 43 and presented in graphical format in figure 39. With the exception of the apparent reduction in faulting with age, table 43 shows a number of logical trends. The peculiar trend exhibited in faulting may be attributable to possible differences in the environmental conditions (temperature and moisture) under which the measurements were taken.

Although the absolute faulting shown in table 43 may not be meaningful, relative faulting does show definite trends. Both 1987 and 1992 data show that faulting is less in the sections provided with plastic-coated dowels than the sections with standard (painted and greased) noncoated dowels. In addition, the faulting of the nondoweled section (OH 1-5) is significantly greater than that of the doweled sections, indicating the effectiveness of dowel bars in reducing faulting.

The sections constructed on ATB show fewer deteriorated transverse cracks and slightly less faulting than the sections constructed on aggregate base. This may speak to the ATB's greater resistance to moisture and erosion. Although one ATB section







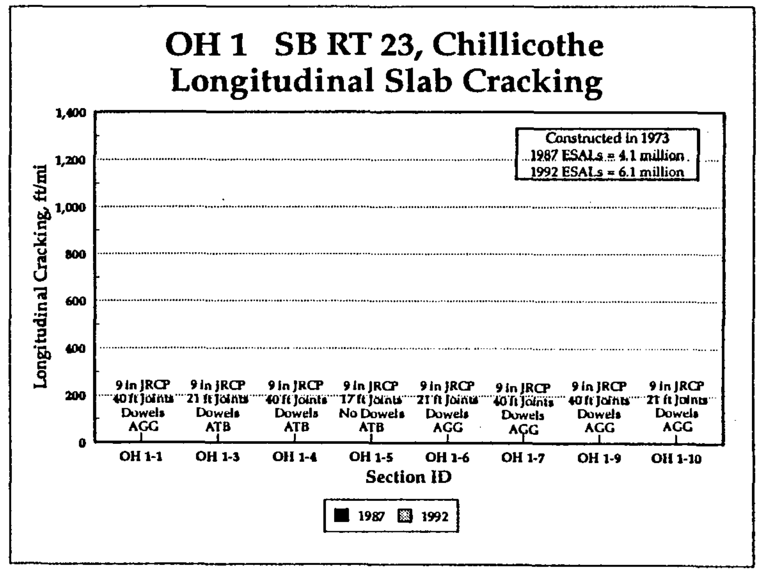


Figure 39. Performance summary for OH 1.

Table 43. Summary of 1987 and 1992 outer lane performance data for OH 1.

			7.5-in (190-mm) Aggregate Base		4-in (102-mm) ATB (5.7%, AC-20)	
			1987 Survey	1992 Survey	1987 Survey	1992 Survey
Age at Survey, years			14	19	14	19
ESAL's, millions			4.1	6.1	4.1	6.1
9-in (229-mm) JRCP 0.09% Steel 21-ft (6.4-m) Jts	1.25-in (32-mm) Dowels (paint/grease)		1-10	1-10	1-3	1-3
		Faulting, in	0.10	0.03	0.06	0.03
		Det. Tr. Crks/mi	0	168	0	0
		Long. Cracks, ft/mi	0	0	0	0
		% Joints Spalled	0	0	13	13
		PSR	4.2	—	4.2	—
		IRI, in/mi	—	182	—	152
	1.25-in (32-mm) Plastic-Coated Dowels		1-6	1-6		
		Faulting, in	0.03	0.01		
		Det. Tr. Crks/mi	31	220		
		Long. Cracks, ft/mi	0	0		
		% Joints Spalled	0	0		
		PSR	4.2	—		
		IRI, in/mi	—	196		
9-in (229-mm) JRCP 0.09% Steel 40-ft (12.2-m) Jts	1.25-in (32-mm) Dowels (paint/grease)		1-1 1-9	1-1 1-9	1-4	1-4
		Faulting, in	0.13 0.14	0.02 0.07	0.07	0.02
		Det. Tr. Crks/mi	0 106	88 251	29	132
		Long. Cracks, ft/mi	0 0	0 0	0	0
		% Joints Spalled	0 0	0 0	0	0
		PSR	4.2 4.2	— —	4.1	—
		IRI, in/mi	— —	224 154	—	156
	1.25-in (32-mm) Plastic-Coated Dowels		1-7	1-7		
		Faulting, in	0.07	0.01		
		Det. Tr. Crks/mi	235	279		
		Long. Cracks, ft/mi	0	0		
		% Joints Spalled	0	0		
		PSR	4.2	—		
		IRI, in/mi	—	135		
9-in (229-mm) JPCP 17-ft (5.2-m) Skewed Jts	No Dowels					1-5
		Faulting, in				0.13
		Det. Tr. Crks/mi				0
		% Slabs Cracked				0
		Long. Cracks, ft/mi				0
		% Joints Spalled				0
		PSR				—
		IRI, in/mi				150

No section contains a subbase course.

1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km

did exhibit some spalling, it was not a significant amount. Interestingly, this section (the only section exhibiting spalling) contained standard (painted and greased) noncoated dowel bars.

The effect of joint spacing on JRCP performance can be evaluated from this project. As expected, the short-jointed (21-ft [6.4-m]) JRCP designs showed fewer deteriorated transverse cracks than the 40-ft (12.2-m) JRCP sections. The 17-ft (5.2-m)

JPCP section shows no deteriorated transverse cracks. The sections with short joint spacing experience less slab movement and lower curling stresses, and also contain more reinforcing steel per unit of slab length even though the percentage of steel is the same for both the long-and short-jointed sections. That is, the 0.09 percent steel is more effective in keeping the cracks tight for the shorter slabs (with less movement) than for the longer slabs. The 0.09 percent steel is probably inadequate for the sections with the longer joint spacing.

Roughness and Serviceability

Figure 40 shows the roughness measurements recorded for the OH 1 sections. Overall, most of the sections are fairly smooth, with the sections constructed on ATB providing the smoothest riding surfaces. Three sections stand out as being particularly rough (OH 1-1, OH 1-6, OH 1-10), all of which show a significant amount of deteriorated transverse cracking. Because these sections are extremely short (typically less than 200 ft [61.0 m]), serviceability ratings were not recorded.

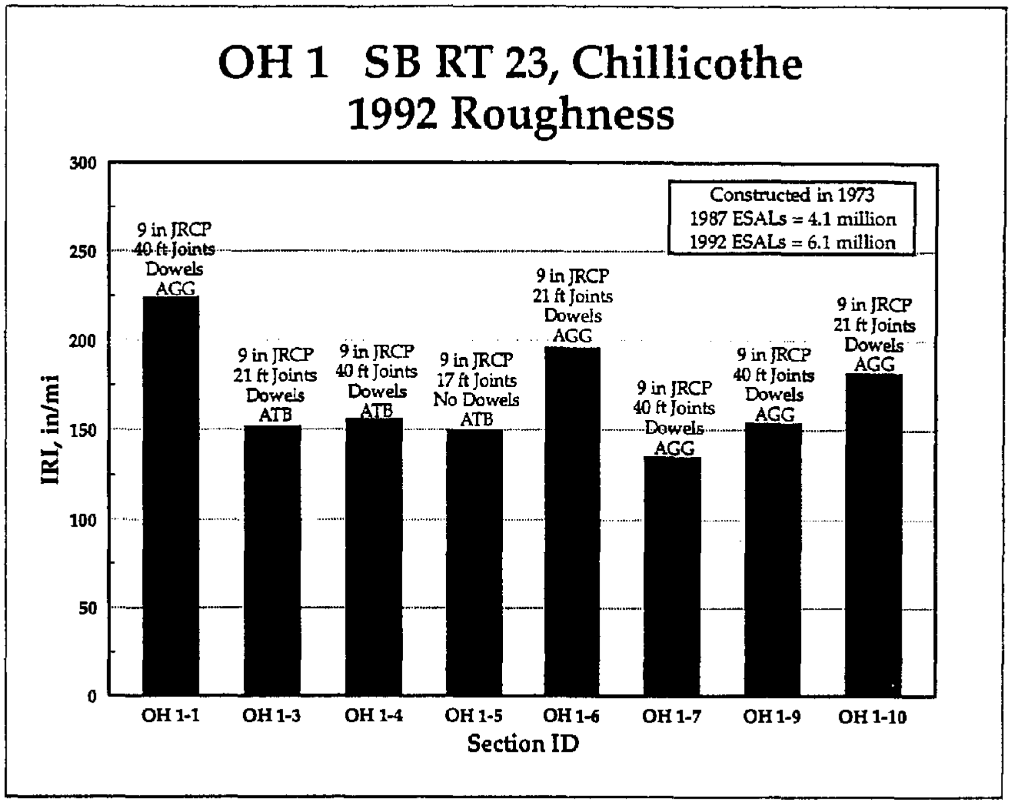


Figure 40. Roughness measurements for OH 1.

5-Year Performance Trends

The 5-year performance trends for the OH 1 sections are shown in figure 41. As mentioned before, the joint faulting measurements for these sections are somewhat anomalous in that the faulting appears to have decreased since 1987. Again, this can be attributed to the ambient temperature and moisture conditions at the time the measurements were taken or to the precision characteristics of the fault-measuring device used on this particular roadway.

The amount of deteriorated transverse cracks increased for every section since the 1987 survey. Significant increases were noted for both 21-ft (6.4-m) and 40-ft (12.2-m) slabs constructed on aggregate base. Many of the cracks on these sections that were rated as low severity in 1987 had deteriorated over the 5-year period and are now either medium or high severity.

Only one section exhibits joint spalling, and the amount of spalling has not changed since 1987. No longitudinal cracking has occurred on any of the sections.

Summary

The performance of the OH 1 sections is summarized below:

- Although the 1987 and 1992 faulting measurements are in conflict, the relative faulting levels between designs indicate less faulting for the sections containing plastic coated dowel bars than the sections containing standard (painted and greased) noncoated dowels. The lone section without dowel bars exhibited a substantial amount of faulting.

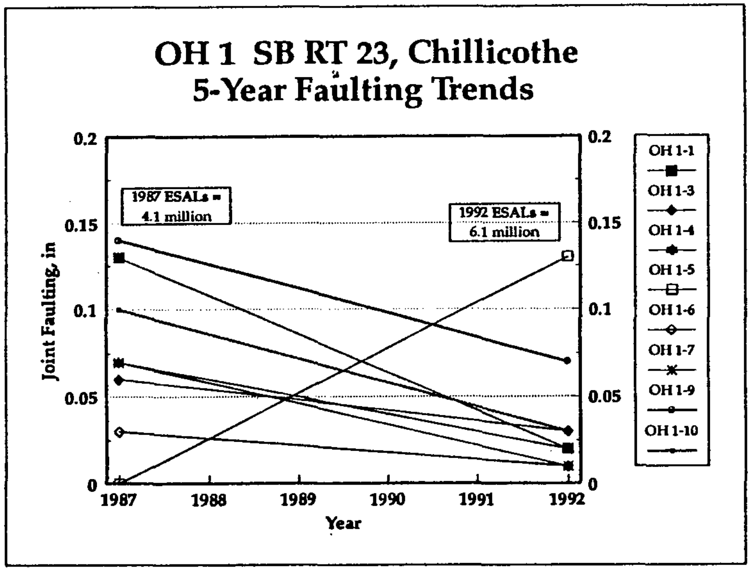
- The sections constructed on ATB showed fewer deteriorated transverse cracks and slightly less faulting than those sections constructed on aggregate base. The ATB, which contained 5.7 percent AC-20, apparently was less susceptible to moisture and erosion than the aggregate base course.

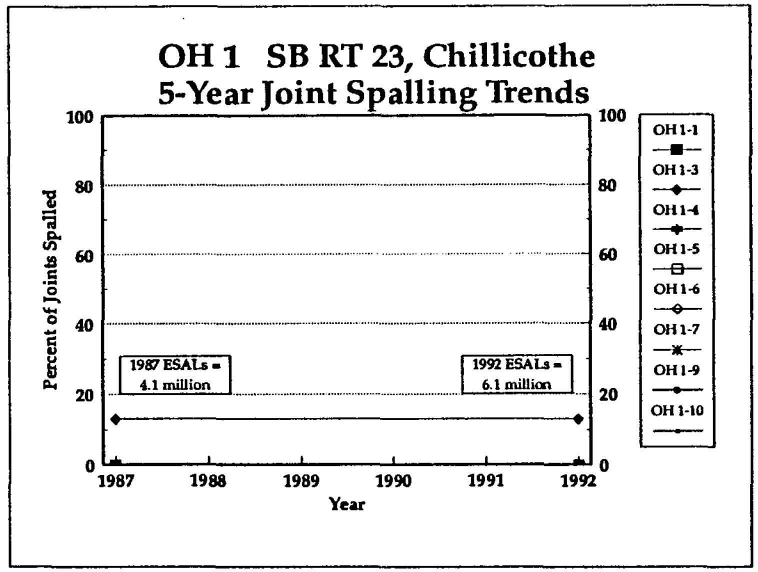
- The short-jointed (21-ft [6.4-m]) JRCP sections showed fewer deteriorated transverse cracks than the 40-ft (12.2-m) JRCP sections (each containing 0.09 percent steel). Similarly, the short-jointed (17-ft [5.2-m]) JPCP design showed no slab cracking.

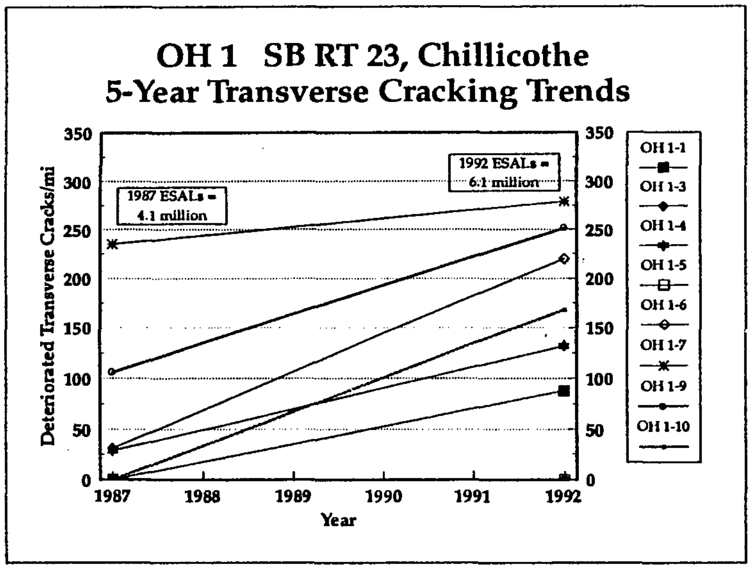
- Overall, the sections constructed on ATB are smoother than those constructed on aggregate base. The sections showing the most roughness are those with a substantial amount of deteriorated transverse cracks. Because of the short length of these sections, serviceability ratings were not conducted.

- Over the 5-year period, every section (with the exception of the JPCP design) developed additional transverse slab cracking. The most significant increases occurred for the sections constructed on aggregate base.

- Only one section exhibited transverse joint spalling, and none of the sections displayed any longitudinal cracking.







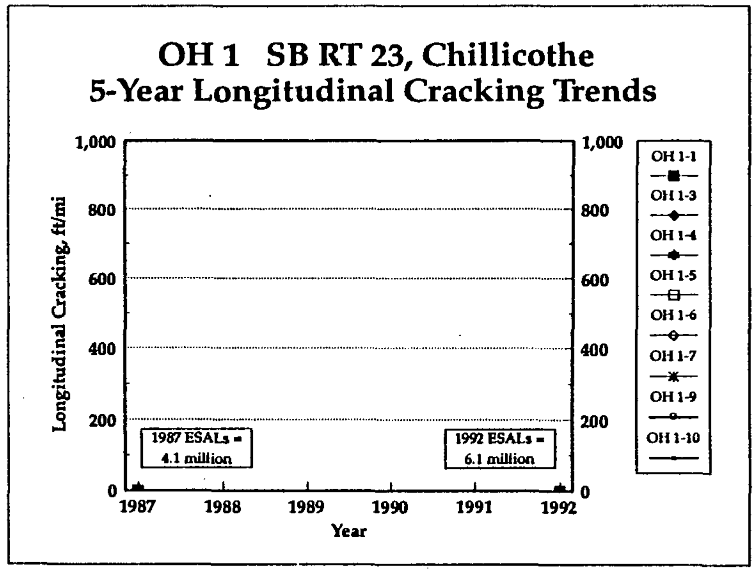


Figure 41. 5-year performance trends for OH 1.

Ohio 2 (S.R. 2, Vermilion)

In 1974, an experimental project was constructed on State Route 2 near Vermilion in order to study the factors that influence the development of D-cracking.[32] The factors examined include the type and maximum size of coarse aggregate, the effect of a pavement drainage system, and other pavement slab design and material factors. The project contains 104 test sections, each about 240-ft (73.2-m) long. Of the 104 sections, two were included in the original 1987 study. In the 1992 survey, a total of 52 sections (including the original 2 surveyed in 1987) were evaluated. Of the 52 sections, 36 were surveyed manually and the rest were surveyed by the automated method only. The data from the automated survey were used to supplement the manual survey data. The reduced design matrix for this project, showing only the sections evaluated under this study, is provided in table 44.

The diversity of designs tested at OH 2 allows the evaluation of a number of different design features, including:

- Edge drains and daylighting.
- Joint sealing and sealant type.
- Joint spacing.
- Maximum coarse aggregate size.
- Coarse aggregate source.
- Pavement type.
- Base type.

However, a meaningful evaluation of some of the above design features (for example, joint sealing) may not be possible because of the extremely short lengths of the sections (about 240 ft [73.2 m] on the average).

Traffic Loadings

This project is a four-lane, divided highway. The stretch of S.R. 2 at the project site is currently subjected to a two-way ADT of 24,900 vehicles per day, including 12 percent heavy trucks. Through 1987, OH 2 had sustained an estimated cumulative traffic of approximately 3.5 million ESAL's. The estimated number of ESAL applications in the outer traffic lane of this roadway through 1992 is 6.5 million ESAL's.

Performance

Tables 45 and 46 summarize the performance of the jointed concrete pavement sections at OH 2. This information is also presented graphically in figures 42 through 44. The performance of the CRCP sections is summarized in table 47. Sections without faulting data represent those where the joints were spalled to bad to measure or those that were surveyed using PASCO.

Table 44. Experimental design matrix for selected sections on OH 2.

		20-ft (6.1-m) JPCP				40-ft (12.2-m) JRCP									60-ft (18.3-m) JRCP		9-in (229-mm) CRCP	
		No Drains		Drains		No Drains			Daylighted			Drains			Drains		No Drains	Drains
		Max. Agg. Size		Max. Agg. Size		Max. Agg. Size			Max. Agg. Size			Max. Agg. Size			Max. Agg. Size		Max. Agg. Size	Max. Agg. Size
Base Type	Seal Type	0.5 in	1.5 in	0.5 in	1.5 in	0.5 in	1.0 in	1.5 in	0.5 in	1.0 in	1.5 in	0.5 in	1.0 in	1.5 in	0.5 in	1.5 in	1.5 in	1.5 in
None	None																	
None	HP	2-1 2-4	2-2 2-3															
None	Pref																	
6-in (152-mm) AGG*	None				2-13			2-73			2-59			2-22				
6-in (152-mm) AGG*	HP			2-17	2-12	2-75	2-74	2-69 2-72§	2-55	2-56	2-54 2-57	2-24	2-23	2-20 2-21	2-18	2-11		
6-in (152-mm) AGG*	Pref				2-14						2-58					2-9		
4-in (102-mm) ATB (4-7%)	None							2-42§						2-53§			2-47	2-48
4-in (102-mm) ATB (4-7%)	HP					2-44§		2-43 2-45§				2-50§		2-49 2-51				
4-in (102-mm) ATB (4-7%)	Pref							2-41§						2-52§				
4-in (102-mm) CTB (4.4%)	None							2-93§						2-104§			2-98	2-99
4-in (102-mm) CTB (4.4%)	HP					2-95§		2-94 2-96§				2-101§		2-100 2-102				
4-in (102-mm) CTB (4.4%)	Pref							2-92§						2-103§				

* OH 2-9, 2-11, 2-12, 2-13, 2-14, 2-17, and 2-18 have 4- to 8.5-in tapered base.
§ PASCO survey only.
OH 2-1 to 2-4 are 15-in slabs (skewed joints) with no dowels, others are 9-in slabs with 1.25-in dowels.
OH 2-1, 2-2, 2-11, and 2-43 have PCC shoulders, others have AC shoulders.

1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km

Table 45. Summary of 1992 outer lane performance data for OH 2 sections with aggregate base.

			20-ft (6.1-m) JPCP		40-ft (12.2-m) JRCP				60-ft (18.3-m) JRCP	
			Max Agg Size		Max Aggregate Size				Max Agg Size	
			0.5 in (13 mm)	1.5 in (38 mm)	0.5 in (13 mm)	1.0 in (25 mm)	1.5 in (38 mm)		0.5 in (13 mm)	1.5 in (38 mm)
Age at Survey, years			18	18	18	18	18	18	18	18
ESAL's, millions			6.5	6.5	6.5	6.5	6.5	6.5	6.5	6.5
No Drains	No Joint Seal							2-73		
		Faulting, in						0.05		
		Det. Tr. Crks/mi						132		
		Long. Cracks, ft/mi						0		
		% Joints Spalled						0		
		PSR								
		IRI, in/mi						170		
	HP				2-75	2-74	2-69	2-72		
		Faulting, in			0.01	0.03	0.01	—		
		Det. Tr. Crks/mi			110	66	132	254		
		Long. Cracks, ft/mi			0	0	0	0		
		% Joints Spalled			14	29	29	0		
		PSR						—		
		IRI, in/mi			130	140	128	—		
Daylighted	No Joint Seal						2-59			
		Faulting, in					0.01			
		Det. Tr. Crks/mi					110			
		Long. Cracks, ft/mi					0			
		% Joints Spalled					14			
		PSR								
		IRI, in/mi					147			
	HP				2-55	2-56	2-57	2-54		
		Faulting, in			0.07	0.02	0.04	0.11		
		Det. Tr. Crks/mi			198	132	132	176		
		Long. Cracks, ft/mi			0	0	0	0		
		% Joints Spalled			0	0	29	0		
		PSR								
		IRI, in/mi			190	114	132	168		
	Preform						2-58			
		Faulting, in					0.02			
		Det. Tr. Crks/mi					106			
		Long. Cracks, ft/mi					0			
		% Joints Spalled					17			
		PSR								
		IRI, in/mi					126			
Edge Drains	No Joint Seal			2-13			2-22			
		Faulting, in		0.13			0.06			
		Det. Tr. Crks/mi		102			132			
		Long. Cracks, ft/mi		0			0			
		% Joints Spalled		79			0			
		PSR								
		IRI, in/mi		157			136			
	HP		2-17	2-12	2-24	2-23	2-20	2-21	2-18	2-11
		Faulting, in	0.24		0	0	0.08	0.04	0.17	
		Det. Tr. Crks/mi	317	0	154	132	132	132	418	88
		Long. Cracks, ft/mi	0	220	0	0	0	0	147	0
		% Joints Spalled	0	100	0	17	57	0	0	100
		PSR								
		IRI, in/mi	239	171	161	172	201	180	296	161
	Preform			2-14						2-9
		Faulting, in		0.03						
		Det. Tr. Crks/mi		22						214
		Long. Cracks, ft/mi		0						0
		% Joints Spalled		100						100
		PSR								
		IRI, in/mi		183						154

OH 2-9, 2-11, 2-12, 2-13, 2-14, 2-17, and 2-18 have 4 to 8.5 in tapered base.
OH 2-11 has PCC shoulders, others have AC shoulders.
All sections are 9 in slabs with 1.25 in dowels.

1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km

Table 46. Summary of outer lane performance data for OH 2 sections with no base, ATB, and CTB.

			20-ft (6.1-m) JPCP				40-ft (12.2-m) JRCP					
			No Base				ATB (4-8% AC)			CTB (4.5% Cement)		
			0.5-in (13-mm) Aggregate		1.5-in (38-mm) Aggregate		0.5-in Agg	1.5-in (38-mm) Aggregate		0.5-in Agg	1.5-in (38-mm) Aggregate	
			1992	1992	1992	1992	1992	1992	1992	1992	1992	1992
Age at Survey, years			18	18	18	18	18	18	18	18	18	18
ESAL's, millions			3.5	6.5	3.5	6.5	6.5	6.5	6.5	6.5	6.5	6.5
No Drain	No Seal							2-42			2-93	
		Faulting, in						—			—	
		Det. Tr. Crks/mi						21			90	
		Long. Cracks, ft/mi						0			0	
		% Joints Spalled						33			83	
		PSR						—			—	
		IRI, in/mi						—			—	
No Drain	HP		2-1	2-4	2-2	2-3	2-44	2-43	2-45	2-95	2-94	2-96
		Faulting, in	0.07	0.30	0.08	0.14	—	0.01	—	—	0.05	—
		Det. Tr. Crks/mi	0	0	0	11	234	22	44	535	110	179
		Long. Cracks, ft/mi	158	0	572	148	0	0	0	0	0	0
		% Joints Spalled	0	0	52	96	0	43	0	33	86	34
		PSR					—		—	—		—
		IRI, in/mi	131	93	143	99	—	276	—	—	153	—
No Drain	Preform							2-41			2-92	
		Faulting, in						—			—	
		Det. Tr. Crks/mi						89			21	
		Long. Cracks, ft/mi						0			0	
		% Joints Spalled						0			17	
		PSR						—			—	
		IRI, in/mi						—			—	
Drain	No Seal							2-53			2-104	
		Faulting, in						—			—	
		Det. Tr. Crks/mi						89			106	
		Long. Cracks, ft/mi						0			0	
		% Joints Spalled						0			17	
		PSR						—			—	
		IRI, in/mi						—			—	
Drain	HP						2-50	2-49	2-51	2-101	2-100	2-102
		Faulting, in					—	0.05	0.06	—	0.06	0.18
		Det. Tr. Crks/mi					258	0	22	389	44	88
		Long. Cracks, ft/mi					260	0	0	0	0	0
		% Joints Spalled					11	100	0	0	43	29
		PSR					—			—		
		IRI, in/mi					—	144	145	—	120	166
Drain	Preform							2-52			2-103	
		Faulting, in						—			—	
		Det. Tr. Crks/mi						0			178	
		Long. Cracks, ft/mi						0			0	
		% Joints Spalled						20			16	
		PSR						—			—	
		IRI, in/mi						—			—	

OH 2-9, 2-11, 2-12, 2-13, 2-14, 2-17, and 2-18 have 4 to 8.5 in tapered base.
OH 2-1, 2-2, and 2-45 have PCC shoulders, others have AC shoulders.
OH 2-1 to 2-4 are 15 in slabs (skewed joints) without dowels, others are 9 in slabs with 1.25 in dowels.

1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km

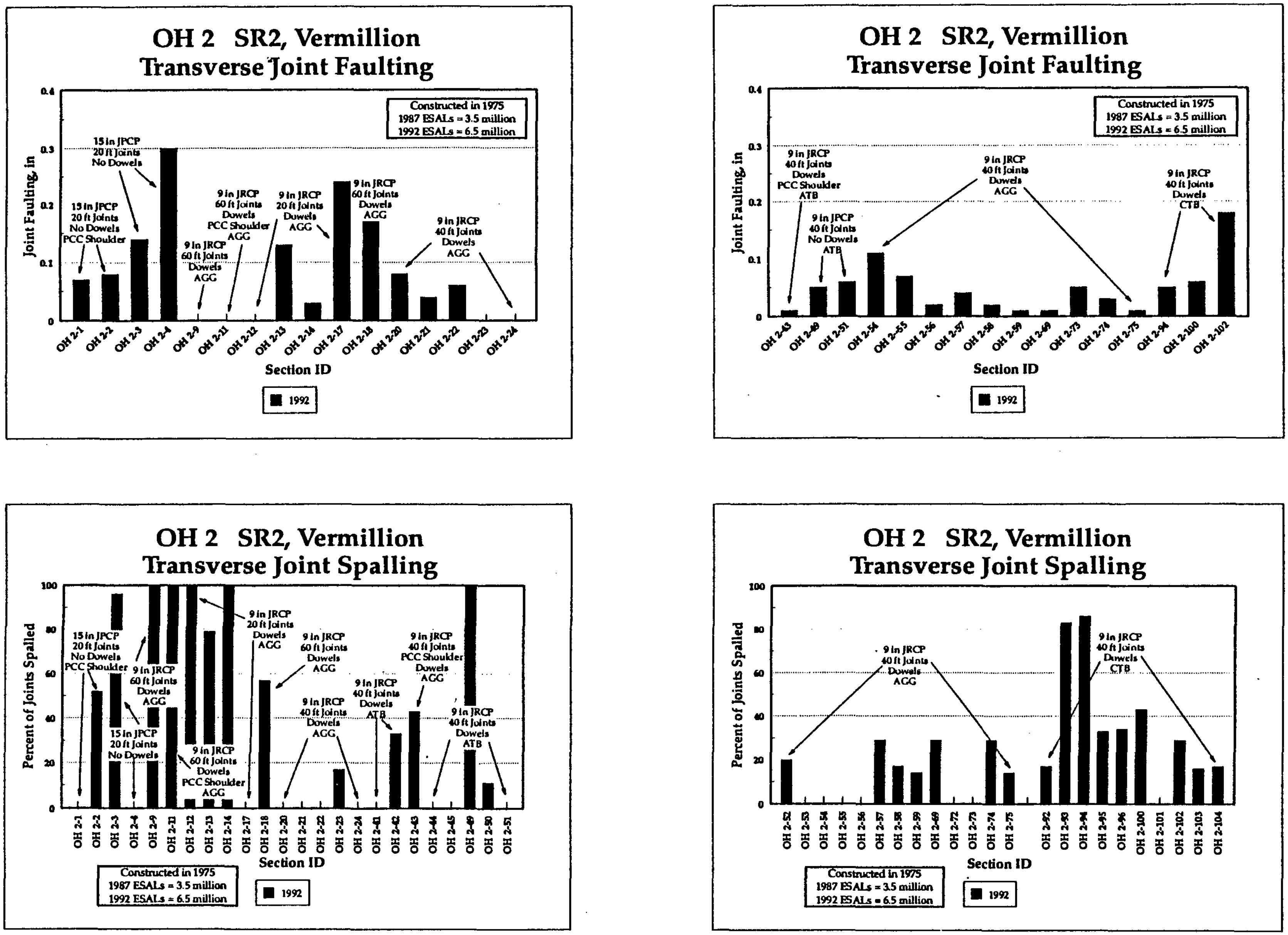
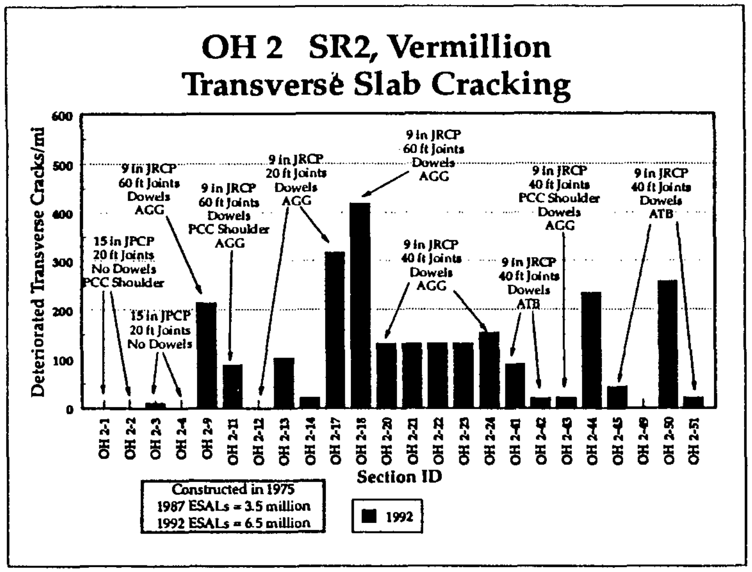
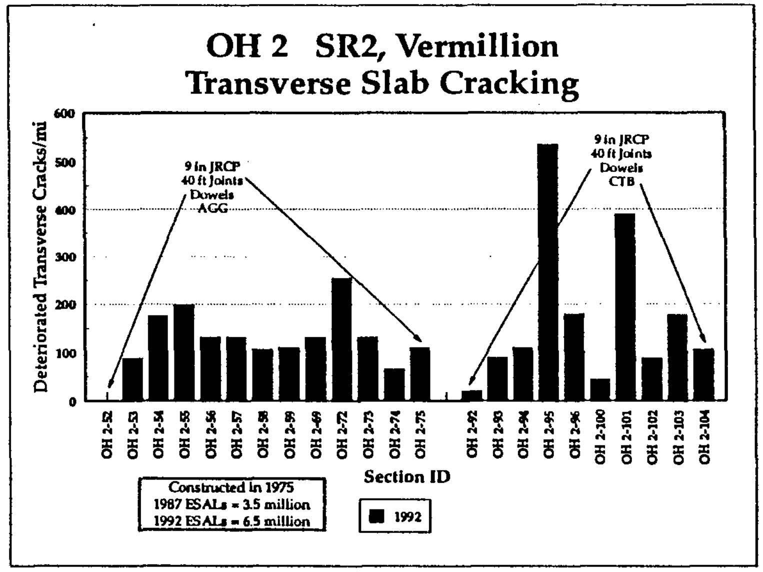
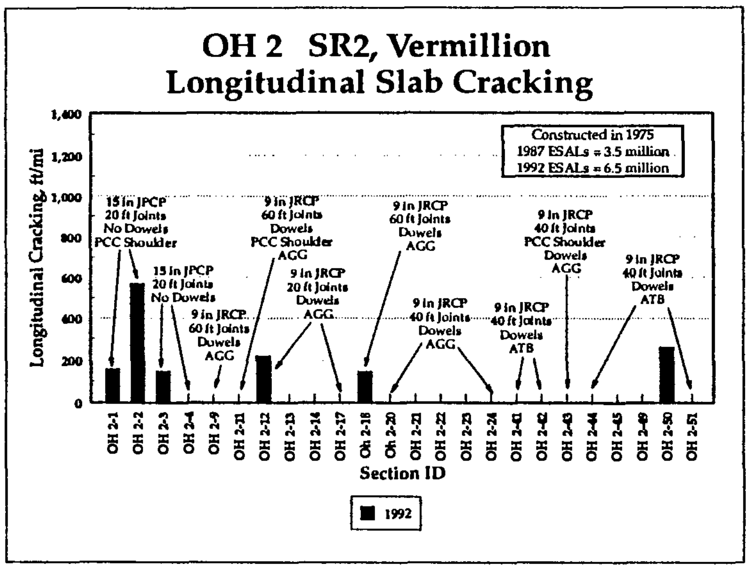


Figure 42. Performance summary of OH 2 sections, joint faulting and spalling.







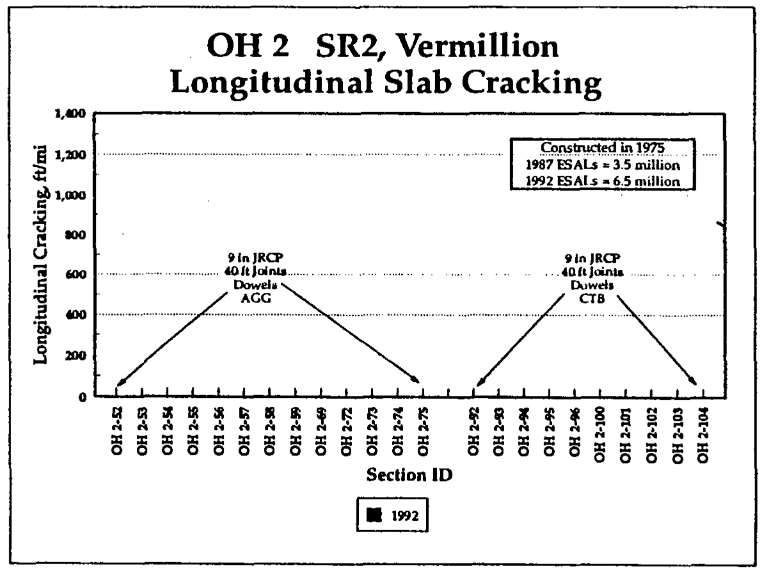
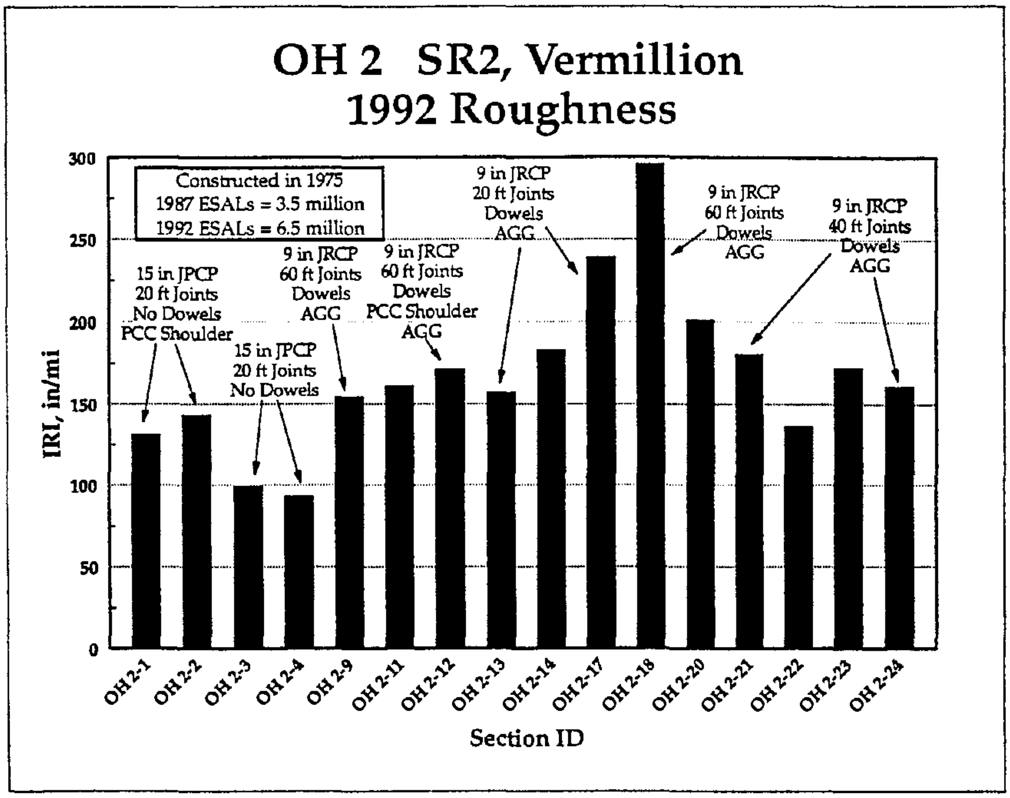


Figure 43. Performance summary of OH 2 sections, transverse and longitudinal cracking.



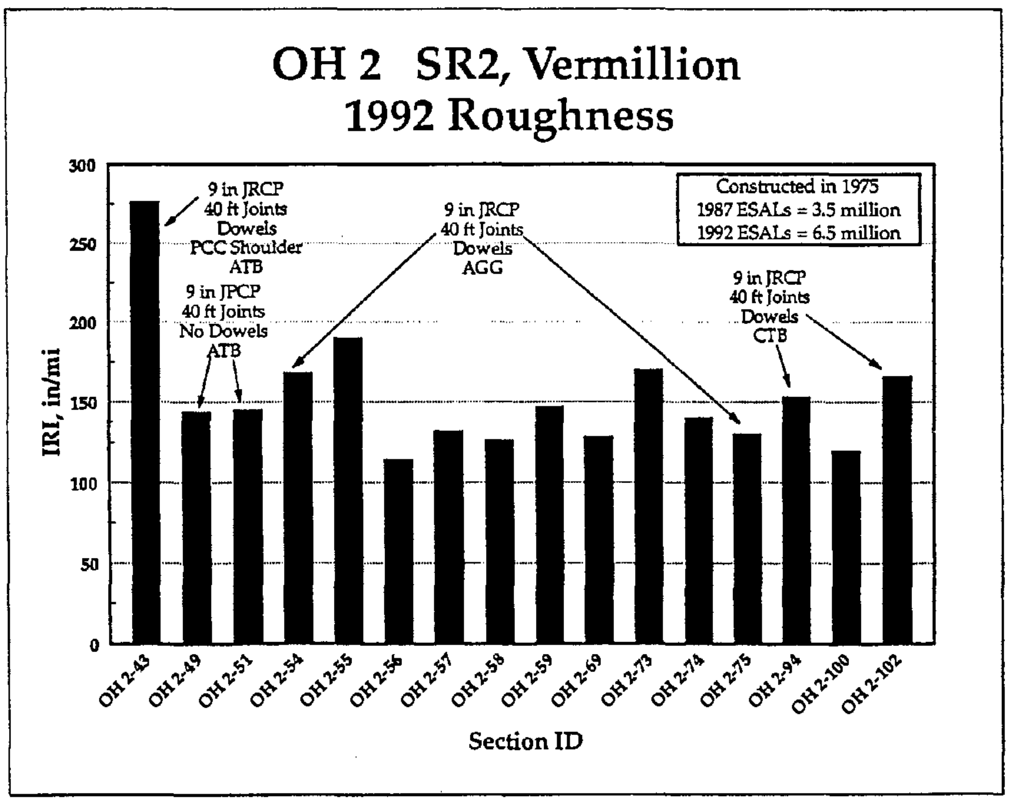


Figure 44. Roughness summary for OH 2 sections.

Table 47. Summary of 1992 outer lane performance data for CRCP sections in OH 2.

	9-in (229-mm) CRCP				
	AGG*	4-in (102-mm) ATB		4-in (102-mm) CTB	
Age at Survey, years	17	17	17	17	17
ESAL's, millions	6.5	6.5	6.5	6.5	6.5
	2-CRC	2-47	2-48	2-98	2-99
Ave Crack Spacing, ft	3.2	3.1	4.1	4.1	6.1
Ave Crack Width, in	0.030	0.056	0.058	0.030	0.030
Det. Tr. Crks/mi	145	238	106	792	449
Long. Cracks, ft/mi					
Failures, no/mi	13	0	0	0	0
PSR					
IRI, in/mi	150	197	144	123	86

* Thickness tapered from 4 to 8.5 in (102 to 216 mm).

1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.6 km

Most of the sections are extensively deteriorated. Many sections exhibit a considerable amount of wide, transverse cracks that are severely spalled and faulted. Figure 42 does not indicate joint faulting to be a significant problem for most sections (with a few exceptions), but faulting of the transverse cracks is severe. Faulting levels of 0.20 to 0.25 in (5 to 6 mm) are common on the deteriorated cracks, with extreme faulting values in excess of 0.5 in (13 mm). Transverse joint spalling ranges from 0 to 100 percent, depending on the type and maximum size of aggregate used in the section and joint spacing. Spalling is also prevalent on many of the high-severity cracks.

The best performers are the 40-ft (12.2-m) JRCP sections with large aggregates, edge drains, and constructed on ATB (OH 2-49 and 2-51). These sections have very little faulting and deteriorated cracks. The CTB sections also provided good performance.

In terms of joint spalling, the 60-ft (18.3-m) JRCP sections constructed with 1.5-in (38-mm) maximum size aggregate (OH 2-9 and OH 2-11) exhibited the worst performance. These sections are so badly spalled that no faulting measurements could be taken. The spalling in the 1.5-in (38-mm) aggregate sections are consistently more severe than the 0.5-in (13-mm) aggregate sections. The principal cause of joint spalling at OH 2 is D-cracking. The reduced maximum aggregate size was one of the measures being tested at this project to mitigate D-cracking, and it appears to be effective in reducing spalling associated with D-cracking. On the 60-ft (18.3-m) JRCP sections with 1.5-in (38-mm) aggregate, both the excessive horizontal movements (due to long joint spacing) and D-cracking appear to have contributed to aggravate the spalling problem.

Pumping is a severe problem in the sections placed without a subbase (OH 2-1 through OH 2-4), even on those sections constructed 15-in (381-mm) thick. Every section was placed on a clayey subgrade with poor permeability. Although it had been at least 2 weeks since it had rained locally at the time of survey, these sections were still pumping water.

The effects of base type and maximum aggregate size on deterioration of transverse cracks in JRCP can be observed from tables 45 and 46. Because small maximum-size aggregate do not provide adequate load transfer through aggregate interlock, crack deterioration in JRCP can be expected to be more of a problem for the sections constructed with 0.5-in (13-mm) aggregate than the ones constructed with 1.5-in (38-mm) aggregate. This trend is shown clearly in table 46 for the sections provided with a stabilized base; however, table 45 shows that for the aggregate base sections, the maximum aggregate size is not a dominant factor affecting the crack deterioration, except for very long-jointed (60-ft [18.3-m] joint spacing) sections. On the 40-ft (12.2-m) JRCP sections constructed on aggregate base, no significant difference in crack deterioration is observed among the sections constructed with different maximum-size aggregate.

Compared with the large-aggregate sections constructed on stabilized bases, the transverse crack deterioration is somewhat more severe in the sections constructed on aggregate base; however, the crack deterioration is two to three times greater in the small-aggregate sections constructed on stabilized bases than in any sections constructed on aggregate bases. One possible explanation for this trend is the role that the shear force of the base plays in the deflection load transfer at the cracks. When a load is placed on one side of a crack, the ability of the unloaded side to match the deflection of the loaded side is restricted by the foundation stiffness. In order for the unloaded side to deflect with the loaded side so that no differential movement occurs at the crack in the vertical direction, the foundation resistance must be overcome by the force transferred from the loaded side through aggregate interlock. The stiffer the foundation, the greater the force required. If the magnitude of this required force is excessive, shear failure will occur at the crack face and the crack will rapidly deteriorate. This mechanism may be the cause of the trends observed from tables 45 and 46. Table 46 shows that the crack deterioration is much more severe in the CTB sections (which have the stiffest foundation) than the ATB sections, providing further support for this hypothesis.

Another key variable in this project is the source of the coarse aggregate. Three sources of coarse aggregate are used in the concrete pavement sections included in this study. The three sources and the corresponding sections in which each source is used are as follows:

- Sy2 is from a source known to produce durable material and is included in sections OH 2-21, OH 2-22, OH 2-51, OH 2-54, OH 2-72, OH 2-73, and OH 2-102.

- Sk681 is a natural gravel with a history of D-cracking where a 1.5 in (38 mm) maximum coarse aggregate size is used. It is included in sections OH 2-45 and OH 2-96.

- Mn3 is a crushed carbonate rock commonly associated with D-cracking where 1.0 and 1.5 in (25 and 38 mm) maximum coarse aggregate size is used. It is the most widely used aggregate source for these sections, used in all sections not previously identified.

Sections containing the Sy2 coarse aggregate showed much better performance than sections containing the Mn3 coarse aggregate, especially in terms of reducing pumping and joint spalling. The Sy2 coarse aggregate, all of which had a maximum size of 1.5 in (38 mm), typically had less pumping and joint spalling than corresponding sections using Mn3 coarse aggregate regardless of the maximum aggregate size (0.5, 1.0, and 1.5 in [13, 25, and 38 mm]). Transverse cracking, on the other hand, was typically more extensive on sections containing the Sy2 coarse aggregate. No clear trends of the effects of aggregate source were observed for faulting or roughness. Sections containing the Sk681 coarse aggregate also showed less joint spalling than the sections containing the Mn3 coarse aggregate, although not to the extent of the Sy2 coarse aggregate. Direct comparisons between sections containing Sy2 and Sk681 are not available. These results correspond with laboratory and field results observed during previous studies.(32)

Other than the observations discussed above, no other trends are apparent from tables 45 and 46. Because of severe spalling and faulting, this project was rehabilitation in 1993. As a part of the rehabilitation project, edge drains were installed in all sections that were not originally provided with drainage, and tied concrete shoulders were added. Badly deteriorated parts of the project were reconstructed.

CRCP Performance

The performance of CRCP sections is summarized in table 47. Section 2-CRC is located in the middle of the experimental sections, but it was not constructed as a part of the experimental pavement. In terms of roughness and failures, the CRCP sections, particularly those constructed on CTB, performed better than jointed sections. The sections constructed on ATB sections have fewer deteriorated cracks, but the CTB section are smoother. Since D-cracking is prevalent throughout this project, the deteriorated cracks may or may not be indicative of structural problems. None of the four sections constructed on stabilized bases contain any punchouts or other failures.

Roughness and Serviceability

The roughness data for OH 2 sections are illustrated in figure 44. These sections are extensively deteriorated and very rough. If the IRI is much greater than about 100 in/mi (1.6 m/km), the ride quality on the roadway is generally poor. Most of the sections in OH 2 have IRI greater than 130 in/mi (2.1 m/km), and several sections have IRI greater than 200 in/mi (3.2 m/km). Two of the four sections with IRI greater than 200 in/mi (3.2 m/km) are 0.5-in (13-mm) aggregate sections (OH 2-17, a 20-ft [6.1-m] JPCP, and OH 2-18, a 60-ft [18.3-m] JRCP). Extensive crack deterioration is the cause of the roughness in these sections. D-cracking is the likely cause of the roughness on the other two rough sections (OH 2-20, a 40-ft [12.2-m] JRCP on aggregate base and OH 2-43, a 40-ft [12.2-m] JRCP on ATB). These sections have a considerable amount of joint spalling.

5-Year Performance Trends

Figure 45 illustrates the 5-year performance trends for sections OH 2-2 and OH 2-3 (these were the only two sections that were evaluated in 1987). OH 2-2 and OH 2-3 are 15-in (381-mm) JPCP placed directly on subgrade, with transverse joints spaced at 20-ft (6.1-m) intervals. With the exception of faulting, the performance of the two sections exhibit a logical trend. Faulting could not be measured at a few of the joints in OH 2-2 because of joint spalling. This spalling, together with some measurement error, may be the cause of the apparent decrease in faulting by 0.03 in (0.8 mm) in OH 2-2 shown in figure 45. Because of very thick slabs, fatigue cracking was not expected on these sections, and no additional transverse cracking was observed.

The only significant change since 1987 for the two sections is the increase in joint spalling. Joint spalling in OH 2-2 increased from 15 percent to 52 percent, while joint spalling in OH 2-3 increased from 23 percent to 96 percent. The primary cause of joint spalling at this project is D-cracking. Although the transverse joints in OH 2-3 are sealed with a hot-pour sealant, the joint spalling is greater than that in OH 2-2, which is not sealed. However, the presence of D-cracking, and not incompressible materials in the joints, is the major cause of joint spalling on this project.

Summary

The performance exhibited by the OH 2 project may be summarized as follows:

- Faulting is more severe at deteriorated cracks than at the joints. Faulting of 0.20 to 0.25 in (5 to 6 mm) is common on deteriorated cracks in this project.

- The best performers are the 40-ft (12.2-m) JRCP sections with large aggregates and edge drains constructed on ATB. These sections have very little faulting and deteriorated cracks.

- On stabilized base sections, deterioration of transverse cracks is much more severe on sections constructed with 0.5-in (13-mm) maximum-size aggregate than on the ones constructed with 1.5-in (38-mm) maximum-size aggregate.

- On aggregate base sections, the maximum aggregate size does not appear to affect deterioration of transverse cracks in JRCP.

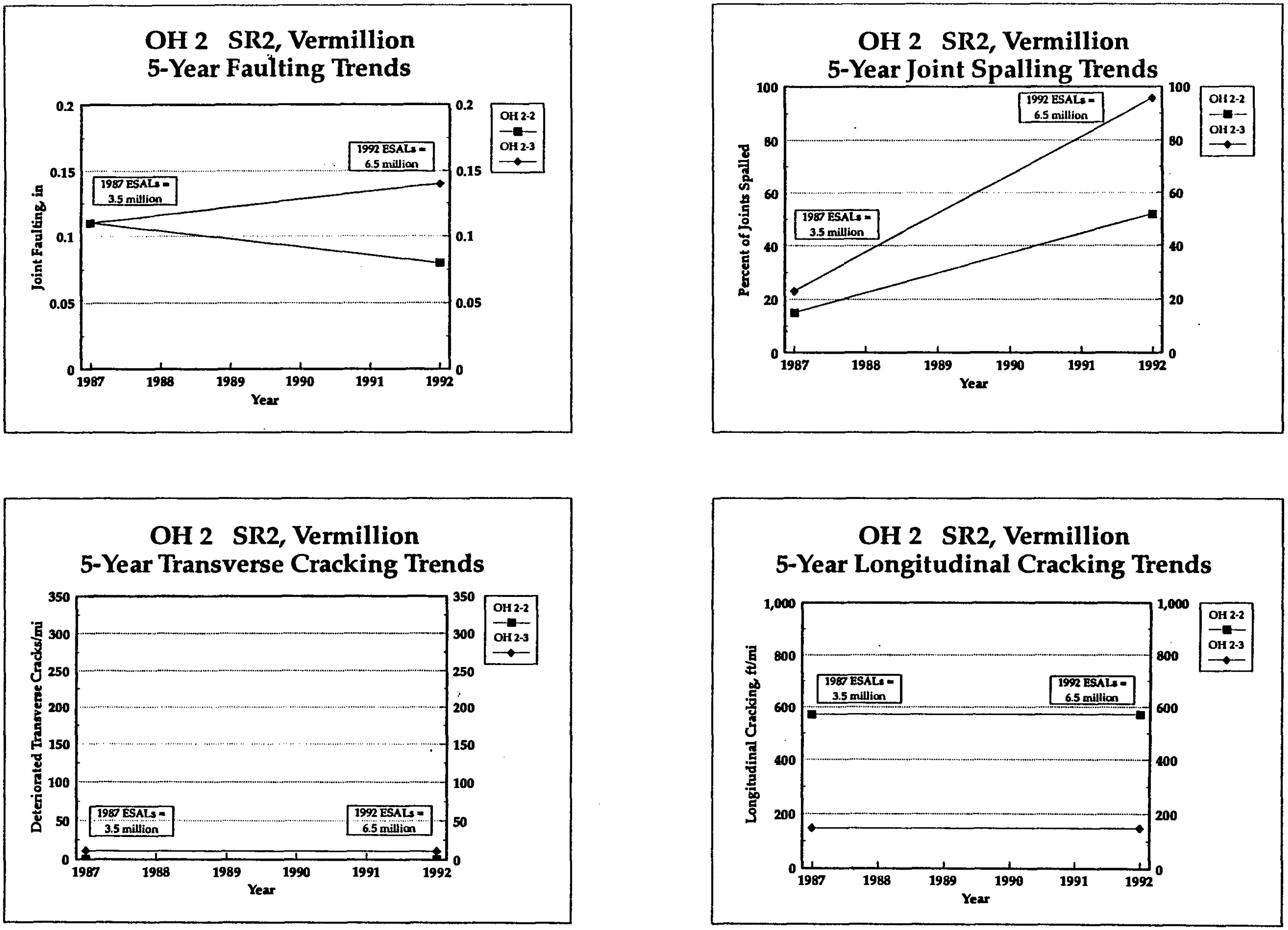


Figure 45. 5-year performance trends for OH 2 sections.

- The 60-ft (18.3-m) JRCP sections constructed with 1.5-in (38-mm) maximum size aggregate (OH 2-9 and OH 2-11) contain the most joint spalling.

- Pumping is a severe problem in the sections placed without a subbase.

<u>Ontario 1 (Hwy. 3N, Windsor)</u>

This project was constructed on Highway 3N near Windsor, Ontario, in 1982. It features the following experimental factors: variations in base type, slab thickness, shoulder type, and surface textures. Four different designs were tested at this site, as shown in table 48.[33,34] Replicate sections were not constructed, but relatively long sections were constructed with each design: three of the four sections are each about 1-mi (1.6-km) long and the remaining section (ONT 1-3) is about 0.5-mi (0.8-km) long. The subgrade at the project site is an AASHTO A-7-6, and all sections contain subdrainage and random, skewed joints placed at 12-13-19-18-ft (3.7-4.0-5.8-5.5-m) intervals. The transverse joints are sealed with a hot-poured joint sealant material and do not contain dowel bars. Two sections contain an LCB, one section contains a PATB (no separator layer beneath base), and one section contains no base course.

Table 48. Experimental design matrix for ONT 1.

	AC Shoulder		PCC Shoulder	
	8-in (203-mm) JPCP, No Dowels	12-in (305-mm) JPCP, No Dowels	7-in (178-mm) JPCP, No Dowels	8-in (203-mm) JPCP, No Dowels
4-in (102-mm) PATB (2% AC)	ONT 1-2			
5-in (127-mm) LCB (7.2% cement)			ONT 1-4	ONT 1-3
No Base		ONT 1-1		

All sections contain longitudinal edge drains and no subbase.

Traffic Loadings

ONT 1 is a well-traveled, two-lane highway in rural Ontario. This road is currently subjected to a two-way ADT of 8,300 vehicles per day, including 8 percent heavy trucks. Through 1987, ONT 1 had sustained approximately 0.9 million ESAL's in each direction. It is now estimated to have accumulated nearly 2.1 million ESAL's through 1992.

Performance

The performance of these sections is summarized in table 49 and presented graphically in figures 46 through 48. The only major difference in performance of the different sections (designs) on this project appears to be the presence of transverse cracking in those sections constructed on LCB. The differences in slab thickness did not appear to be a significant factor affecting performance. The LCB sections also represent the only sections that have tied concrete shoulders (2-ft [0.6-m] wide) at this project. Because the base type is also a significant factor affecting transverse cracking, a meaningful comparison of the effects of shoulder type on pavement performance is not possible with the ONT 1 sections.

Table 49. Summary of 1987 and 1992 outer lane performance data for ONT 1.

		AC Shoulder				PCC Shoulder			
		8-in (203-mm) JPCP		12-in (305-mm) JPCP		7-in (178-mm) JPCP		8-in (203-mm) JPCP	
		1987 Survey	1992 Survey	1987 Survey	1992 Survey	1987 Survey	1992 Survey	1987 Survey	1992 Survey
Age at Survey, years		5	10	5	10	5	10	5	10
ESAL's, millions		0.9	2.1	0.9	2.1	0.9	2.1	0.9	2.1
4-in (102-mm) PATB (2% AC)		1-2	1-2						
	Faulting, in	0.05	0.10						
	Det. Tr. Crks/mi	0	0						
	% Slabs Cracked	0	0						
	Long. Cracks, ft/mi	0	105						
	% Joints Spalled	0	1						
	PSR	3.8	3.9						
	IRI, in/mi	—	135						
5-in (127-mm) LCB (7.2% cement)						1-4	1-4	1-3	1-3
	Faulting, in					0.04	0.13	0.04	0.14
	Det. Tr. Crks/mi					45	48	30	28
	% Slabs Cracked					14	14	9	8
	Long. Cracks, ft/mi					345	516	490	621
	% Joints Spalled					0	0	0	0
	PSR					3.7	3.9	3.8	3.9
	IRI, in/mi					—	164	—	147
No Base				1-1	1-1				
	Faulting, in			0.05	0.11				
	Det. Tr. Crks/mi			0	0				
	% Slabs Cracked			0	0				
	Long. Cracks, ft/mi			40	40				
	% Joints Spalled			0	0				
	PSR			3.8	3.9				
	IRI, in/mi			—	146				

All sections contain longitudinal edge drains and no subbase.

1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km

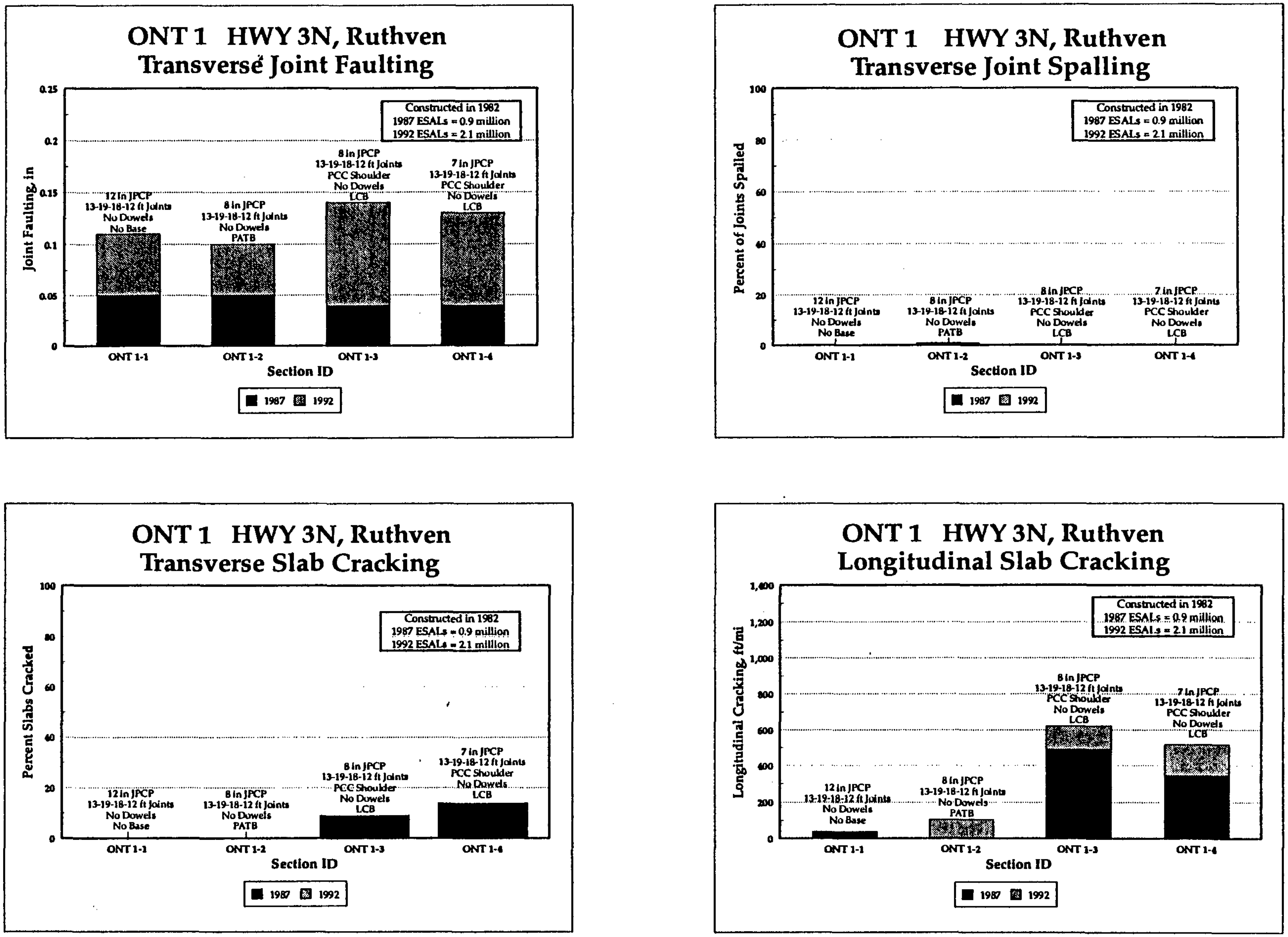
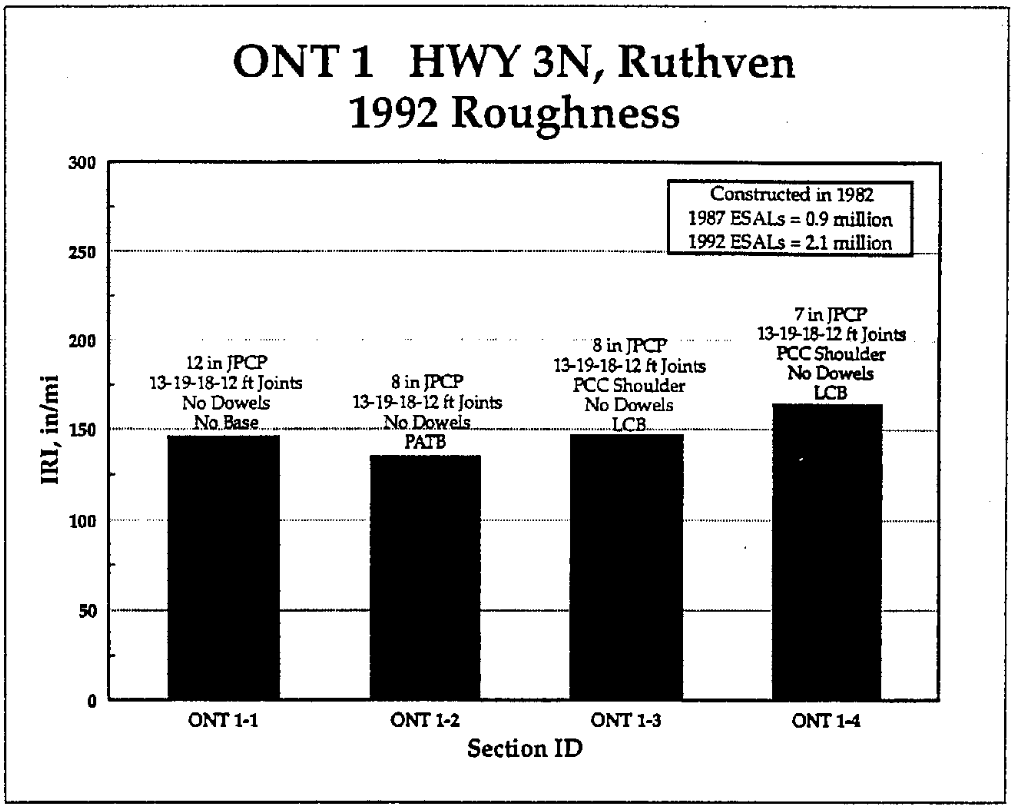


Figure 46. Performance summary for ONT 1 sections.



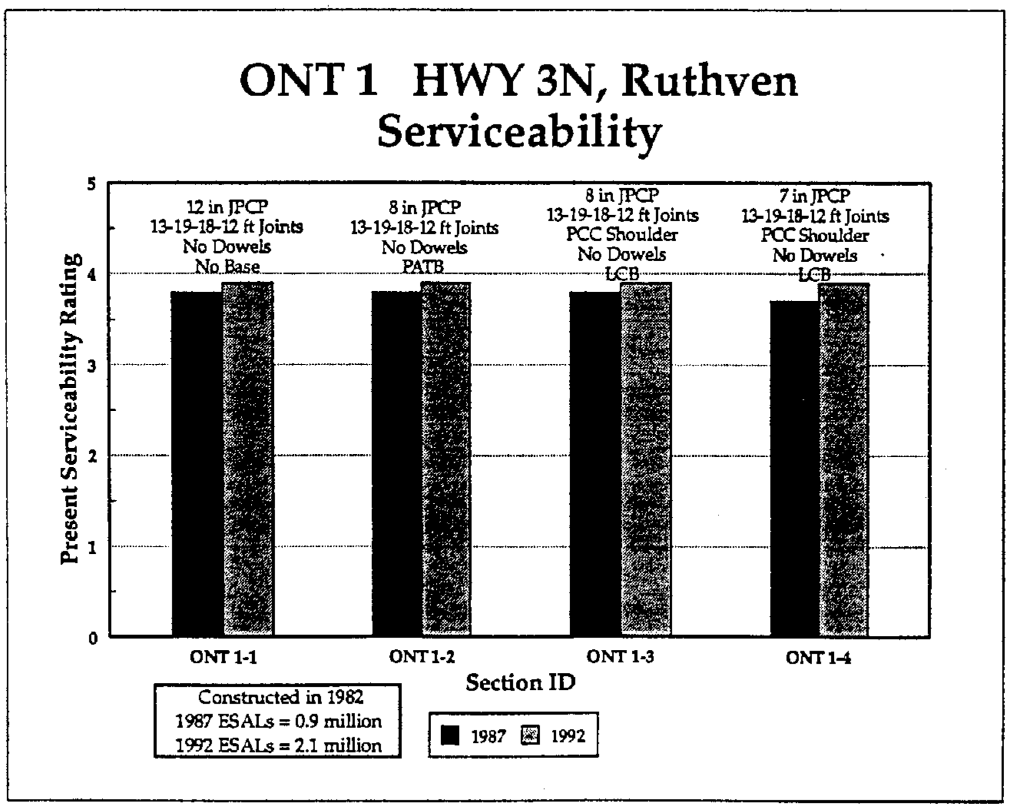


Figure 47. Roughness and serviceability summary for ONT 1 sections.

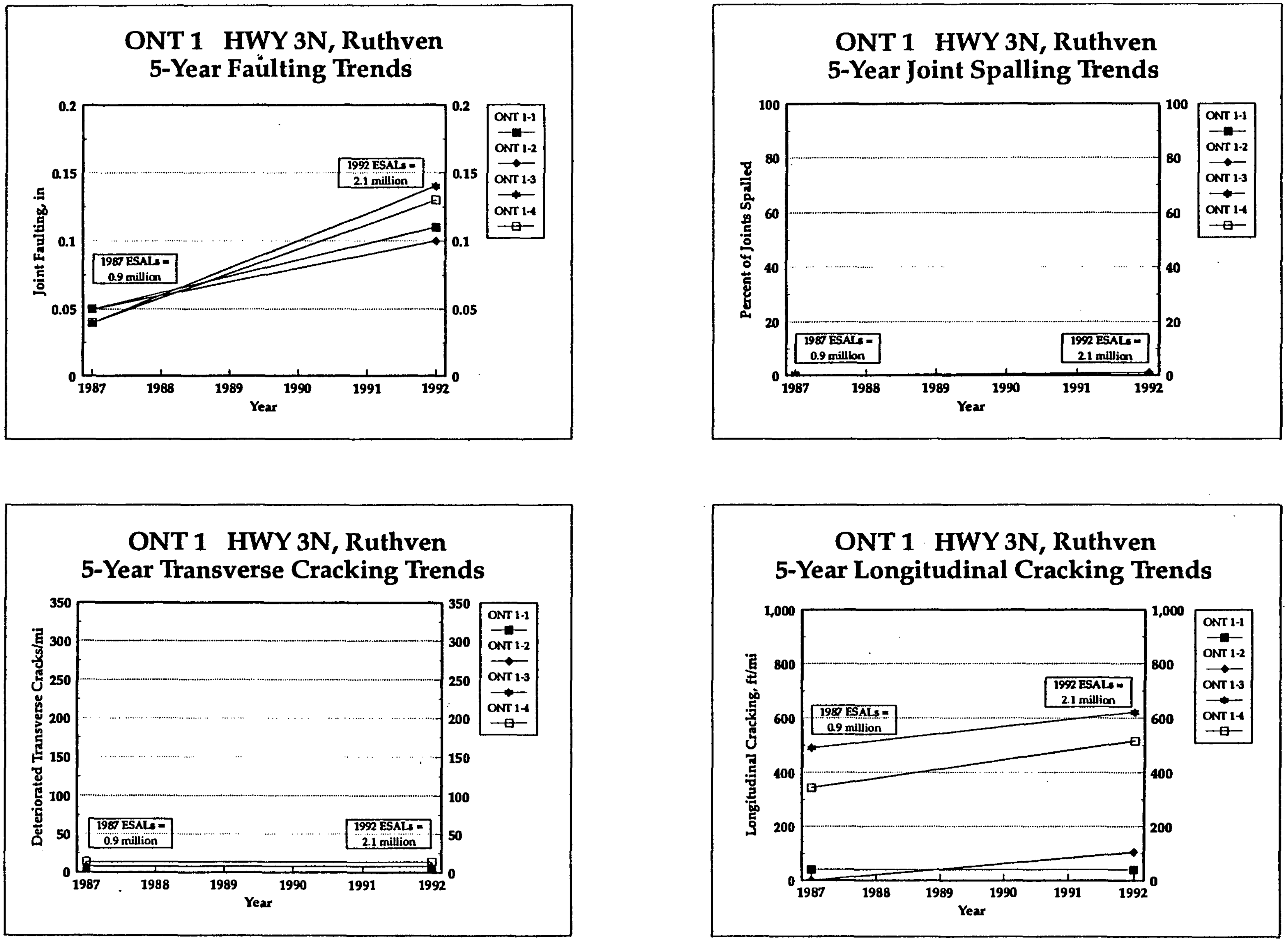


Figure 48. 5-year performance trends for ONT 1 sections.

ONT 1-2 contains narrow strips of partial-depth patches that are about 10-in (254-mm) wide and extend over several slabs. These patches are always in pairs that are about 4 ft (1.2 m) apart. Longitudinal cracking started to develop at the tip of some of these patches in this section. The reason for the presence of the partial-depth patches is not known, but the longitudinal cracking in this section is clearly not related to any of the design factors. The longitudinal cracking in ONT 1-3 and ONT 1-4 occurred very close to the centerline joint, suggesting improper sawing of the centerline joint as the prime suspect for the cause of the cracking.

Roughness and Serviceability

The roughness and present serviceability data for ONT 1 sections are shown in figure 47. As shown in this figure, both roughness and the perceived ride quality on all sections are similar. A fair amount of roughness was observed on these sections. The primary source of roughness in these sections appears to be joint faulting. Based on PSR, the ride quality over these sections does not appear to have deteriorated since 1987. The PSR for the ONT 1 sections are very high for the IRI values shown (average of about 150 in/mi [2.4 m/km]). The average PSR of 3.8 is typically associated with IRI values far less than 100 in/mi (1.6 m/km).

5-Year Performance Trends

The 5-year performance trends for the ONT 1 sections are shown in figure 48. Compared to the condition in 1987, all sections show a significant increase in faulting. The LCB sections (ONT 1-3 and ONT 1-4) showed less faulting in 1987 than sections with no base and a PATB (ONT 1-1 and ONT 1-2); but by 1992, the faulting in the LCB sections surpassed that of the other sections. None of the ONT 1 sections are doweled; hence, aggregate interlock is the only mechanism for load transfer for these sections. If the mechanism described in the evaluation of the OH 2 sections to explain the cause of rapid deterioration of transverse cracks in the stabilized-base sections of is valid, the greater amount of faulting in the sections constructed on stiffer base without positive load transfer is an expected trend.

Additional longitudinal cracking is observed on ONT 1-2, ONT 1-3 and ONT 1-4, but no other joint spalling or transverse cracking occurred over the 5-year period. Judging by the fact that the transverse cracking occurred only in the LCB section and that no further cracking occurred between 1987 and 1992, the loss of support (contact) between the base and the slab due to curling, and not the fatigue damage, appears to be the cause of transverse cracking in ONT 1-3 and ONT 1-4.

Summary

The performance exhibited by this project may be summarized as follows:

- The only major difference in the performance of the different sections on this project is the presence of transverse cracking in the LCB sections. The amount of transverse cracking in these sections remained the same since 1987.

- All sections in ONT 1 exhibit a fair amount of roughness (average IRI of about 150 in/mi [2.4 m/km]), but the perceived ride quality is very high, possibly due to the skewed joints. The average PSR from 1992 survey is 3.8.

- Faulting in all sections increased significantly since 1987 (from an average of 0.05 in [1.3 mm] in 1987 to an average of 0.12 in [3.0 mm] in 1992). The faulting in LCB sections progressed more rapidly than in other sections.

Ontario 2 (Hwy. 427, Toronto)

This project is a single JPCP section located on Highway 427 in Toronto. Highway 427 is a principal access route into downtown Toronto. At the location of the survey section, the highway has four lanes in the direction of survey, and a total of 11 lanes. Constructed in 1971, the section features a 9-in (229-mm) slab constructed on a 6-in (152 mm) CTB with skewed, doweled joints at 12-13-19-18-ft (3.7-4.0-5.8-5.5-m) intervals. Longitudinal edge drains were added in 1982. Design and performance data for ONT 2 are given in table 50.

The two-way ADT on this highway is currently 246,000 vehicles per day, including 10 percent heavy trucks. At the time of 1987 survey, this section had sustained 35.6 million ESAL's. Through 1992, the pavement is estimated to have accumulated 56.0 million ESAL's.

In the 1992 survey, the original survey section and a replicate section were surveyed. A summary of the outer lane performance is shown in table 50. As shown in this table, ONT 2 is in excellent condition. The entire section is virtually free of cracking and spalling, and, although a definite increase in faulting has occurred since 1987 (from 0.01 to 0.05 in [0.2 to 1.3 mm]), it does not represent a performance problem. Considering the traffic volume, pavement structure, environmental condition, this pavement is exhibiting remarkable performance.

Pennsylvania 1 (RT. 66 and RT. 422, Kittanning)

In 1980, an experimental JRCP consisting of bases of varying permeabilities was constructed on Routes 66 and 422 near Kittanning to investigate the performance of the alternative base types.[35] The base types tested include CTB, PATB, uniformly graded aggregate, well-graded aggregate, and dense-graded aggregate. All sections have a 10-in (254-mm) pavement slab with 46.5-ft (14.2-m) joint spacing and 1.25-in (32-mm) epoxy-coated dowels. The subgrade type varies from A-2-4 to A-4. The experimental design for this project is shown in table 51.

The CTB section (PA 1-1) is constructed on Route 422 and is not replicated on Route 66. The only other section constructed on Route 422 is the control section utilizing Pennsylvania's conventional aggregate base design (dense-graded aggregate), which is also replicated on Route 66. All remaining sections were constructed on Route 66 and replicated in both direction of the divided roadway.

Table 50. Design and outer lane performance data for ONT 2.

	ONT 2		
Pavement Type	JPCP		
Year Built	1971		
Thickness, in	9-in (229-mm)		
Joint Spacing, ft	12-13-19-18 (3.7-4.0-5.8-5.5-m)		
Dowel Diameter, in	1.0 (25-mm)		
Base Type	6-in (152-mm) CTB (5% cement)		
Subgrade	AASHTO A-4		
Performance Data	1987 Survey	1992 Survey	
Age at Survey, years	16	21	
ESAL's, millions	35.6	56.0	
	2-1	2-1	2-1a
Faulting, in	0.01	0.05	0.04
Det. Tr. Crks/mi	5	5	5
% Slabs Cracked	3	3	2
Long. Cracks, ft/mi	0	0	0
% Joints Spalled	0	0	3
PSR	3.9	4.0	3.8
IRI, in/mi	—	147	180

1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km

Table 51. Experimental design matrix for PA 1.

	6-in (152-mm) CTB (6 %)	**5-in (127-mm) PATB (2%)**	**8-in (203-mm) Uniform-Graded AGG (Permeable)**	**8-in (203-mm) Well-Graded AGG (Permeable)**	**13-in (330-mm) Dense-Graded AGG**
10-in (254-mm) JRCP (0.09% Steel) 46.5-ft (14.2-m) Joints	PA 1-1	PA 1-2	PA 1-3	PA 1-4	PA 1-5

1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km

Traffic Loadings

Route 422 is a four-lane divided highway with access control. PA 1-1 is located at one end of Route 422, close to where the roadway ends and turns into an exit ramp. The two-way ADT on this road is currently 16,000 vehicles per day, including 4 percent heavy trucks. At the time of 1987 survey, PA 1-1 had sustained an estimated cumulative traffic of approximately 0.6 million ESAL applications. The accumulated ESAL applications on this route through 1992 is 1.1 million.

All other sections at PA 1 are located on Route 66. Route 66 is also a four-lane divided highway, but it is not access controlled. The two-way ADT on this road is currently 17,200 vehicles per day, including 4 percent heavy trucks. At the time of 1987 survey, the PA 1 sections on Route 66 had sustained an estimated cumulative traffic of approximately 0.3 million ESAL applications. Through 1992, it had accumulated approximately 0.8 million ESAL applications.

Performance

A summary of the outer lane performance data for PA 1 is shown in table 52 and presented graphically in figures 49 through 51. The only significant difference in the performance of the different designs is the deterioration of transverse cracks in PA 1-1, the CTB section. This cracking may be expected since stiffer base leads to higher curling stresses in the slab. No section has an appreciable amount of faulting or spalling at the joints.

Roughness and Serviceability

Roughness and serviceability of PA 1 sections are presented in figure 50. The ride quality on PA 1 sections is very good and all sections have similar serviceability. PA 1-3 has slightly higher IRI than other sections and PA 1-5 is slightly lower. The deviation is relatively small, but the cause of the deviation is difficult to understand. The two sections have similar levels of faulting and exhibit no other distresses.

5-Year Performance Trends

The 5-year performance trends of PA 1 sections is presented in figure 51. The only notable changes since 1987 are the slight increase in the amount of deteriorated transverse cracks and faulting in PA 1-1. PA 1-1 is constructed on a CTB, and is the only section with deteriorated transverse cracks. The amount of faulting in all sections is negligible (average of about 0.03 in [0.8 mm]) and faulting in PA 1-1 is also not significant (0.05 in [1.3 mm]), but while faulting in other sections remained the same since 1987, the faulting in PA 1-1 has roughly doubled. No other significant change was observed.

Table 52. Summary of 1987 and 1992 outer lane performance data for PA 1.

		1987 Survey	1992 Survey	1992 Replicate
Age at Survey, years		7	12	12
ESAL's, millions		0.3	0.8	0.8
6-in (152-mm) CTB (6 %)		1-1*	1-1*	
	Faulting, in	0.03	0.05	
	Det. Tr. Crks/mi	0	10	
	Long. Cracks, ft/mi	0	0	
	% Joints Spalled	0	0	
	PSR	4.2	3.9	
	IRI, in/mi	—	134	
5-in (127-mm) PATB (2%)		1-2	1-2	1-2a
	Faulting, in	0.02	0.01	0.03
	Det. Tr. Crks/mi	0	0	0
	Long. Cracks, ft/mi	0	0	0
	% Joints Spalled	0	4	0
	PSR	3.8	4.2	
	IRI, in/mi	—	150	120
8-in (203-mm) Uniform-Graded Aggregate		1-3	1-3	1-3a
	Faulting, in	0.03	0.03	0.03
	Det. Tr. Crks/mi	0	0	0
	Long. Cracks, ft/mi	0	0	0
	% Joints Spalled	0	0	0
	PSR	3.7	4.1	
	IRI, in/mi	—	178	134
8-in (203-mm) Well-Graded Aggregate		1-4	1-4	1-4a
	Faulting, in	0.03	0.03	0.04
	Det. Tr. Crks/mi	0	0	0
	Long. Cracks, ft/mi	0	0	0
	% Joints Spalled	0	0	0
	PSR	4.0	4.0	
	IRI, in/mi	—	159	159
13-in (330-mm) Dense Graded Aggregate		1-5	1-5	1-5a
	Faulting, in	0.03	0.02	0.03
	Det. Tr. Crks/mi	0	0	0
	Long. Cracks, ft/mi	0	0	0
	% Joints Spalled	0	0	0
	PSR	4.0	4.3	
	IRI, in/mi	—	127	149

* Estimated ESAL's to date for PA 1-1 was 0.6 million in 1987 and 1.1 million in 1992.

PA 1-1 has 7-in AGG subbase.
PA 1-2 has 8-in AGG subbase.
PA 1-3 and 1-4 have 5-in AGG subbase.
PA 1-5 has no subbase course.

1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km

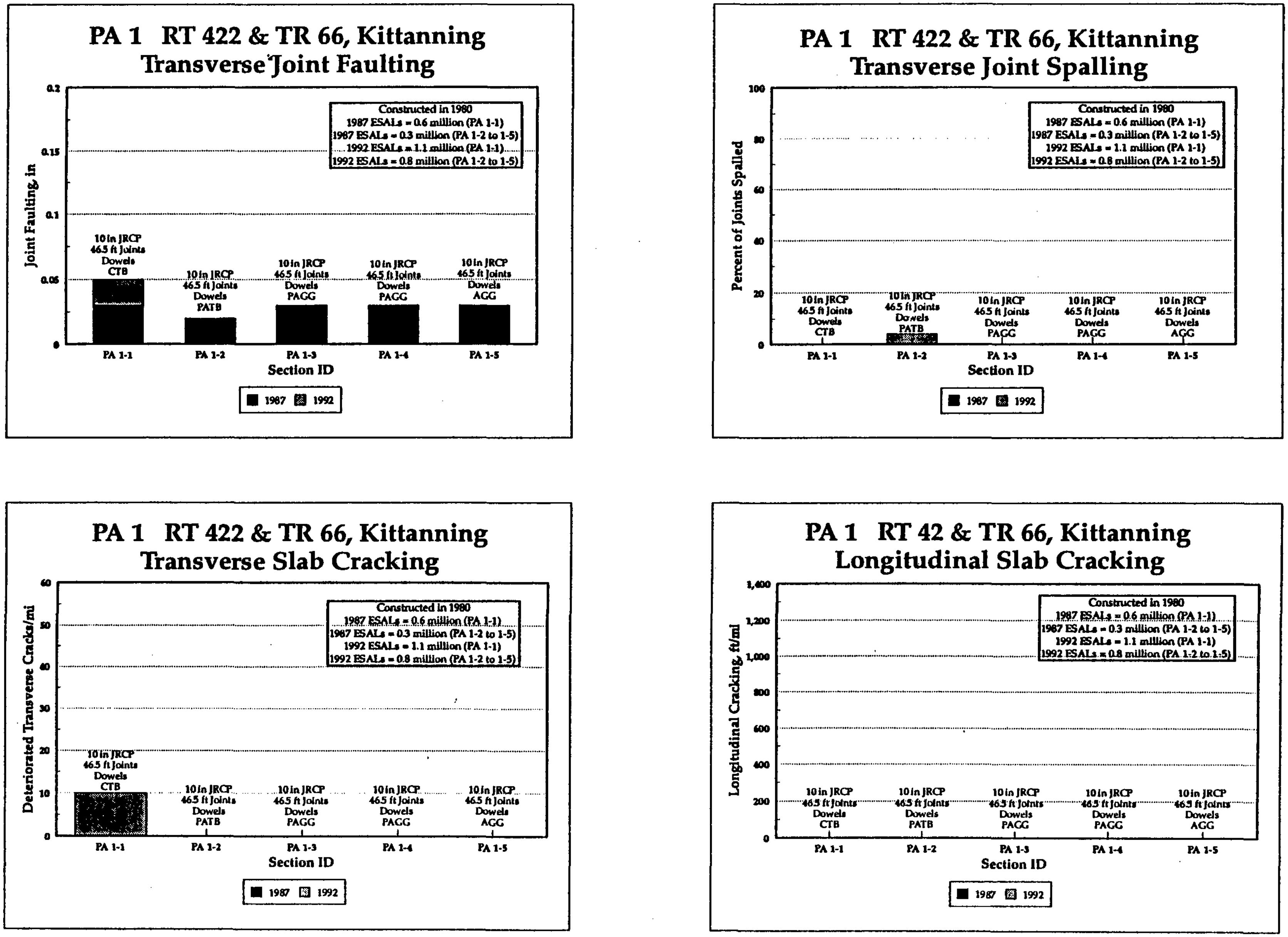
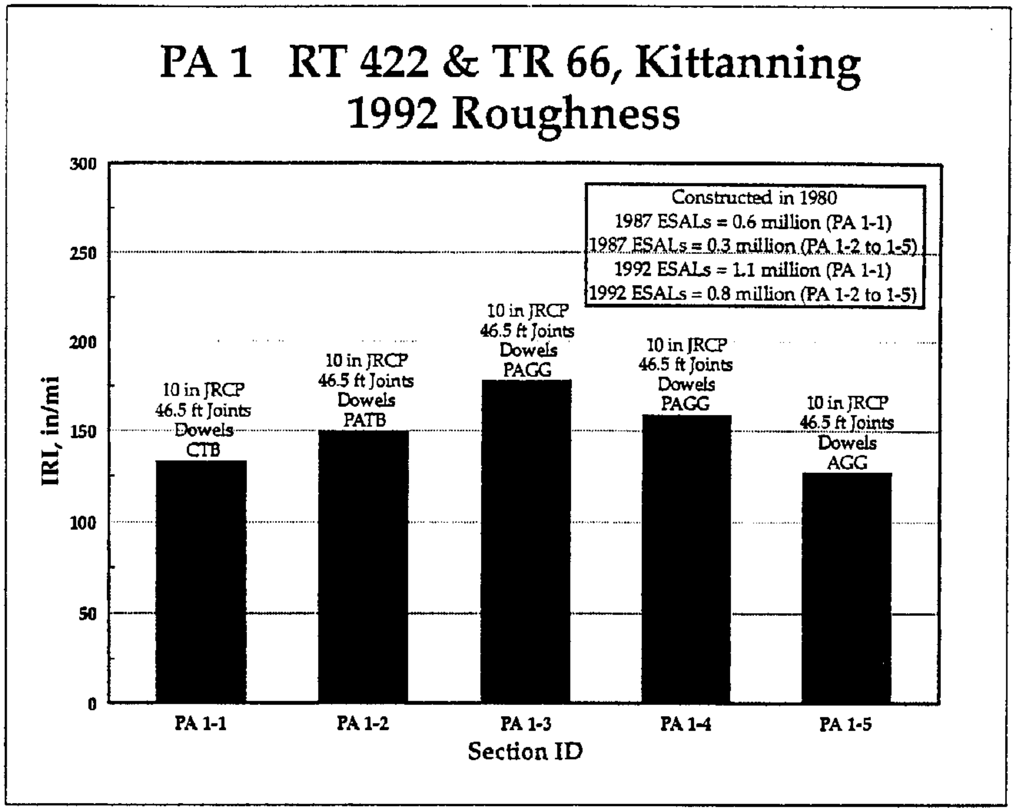


Figure 49. Performance summary for PA 1 sections.



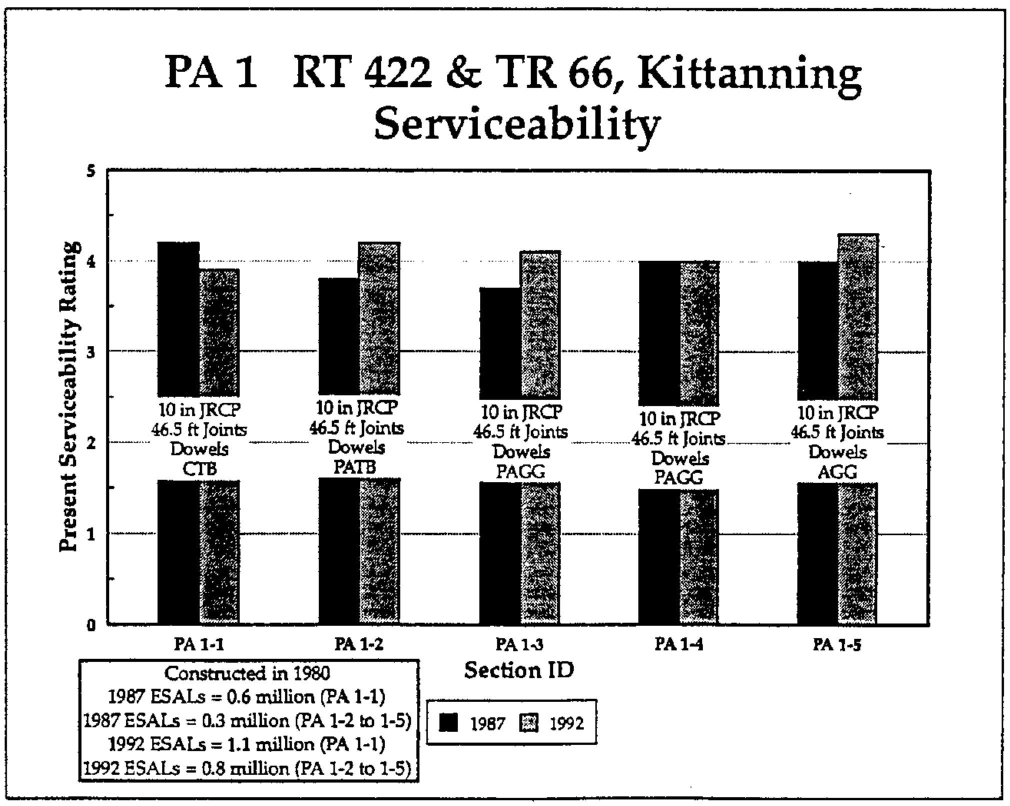


Figure 50. Roughness and serviceability for PA 1 sections.

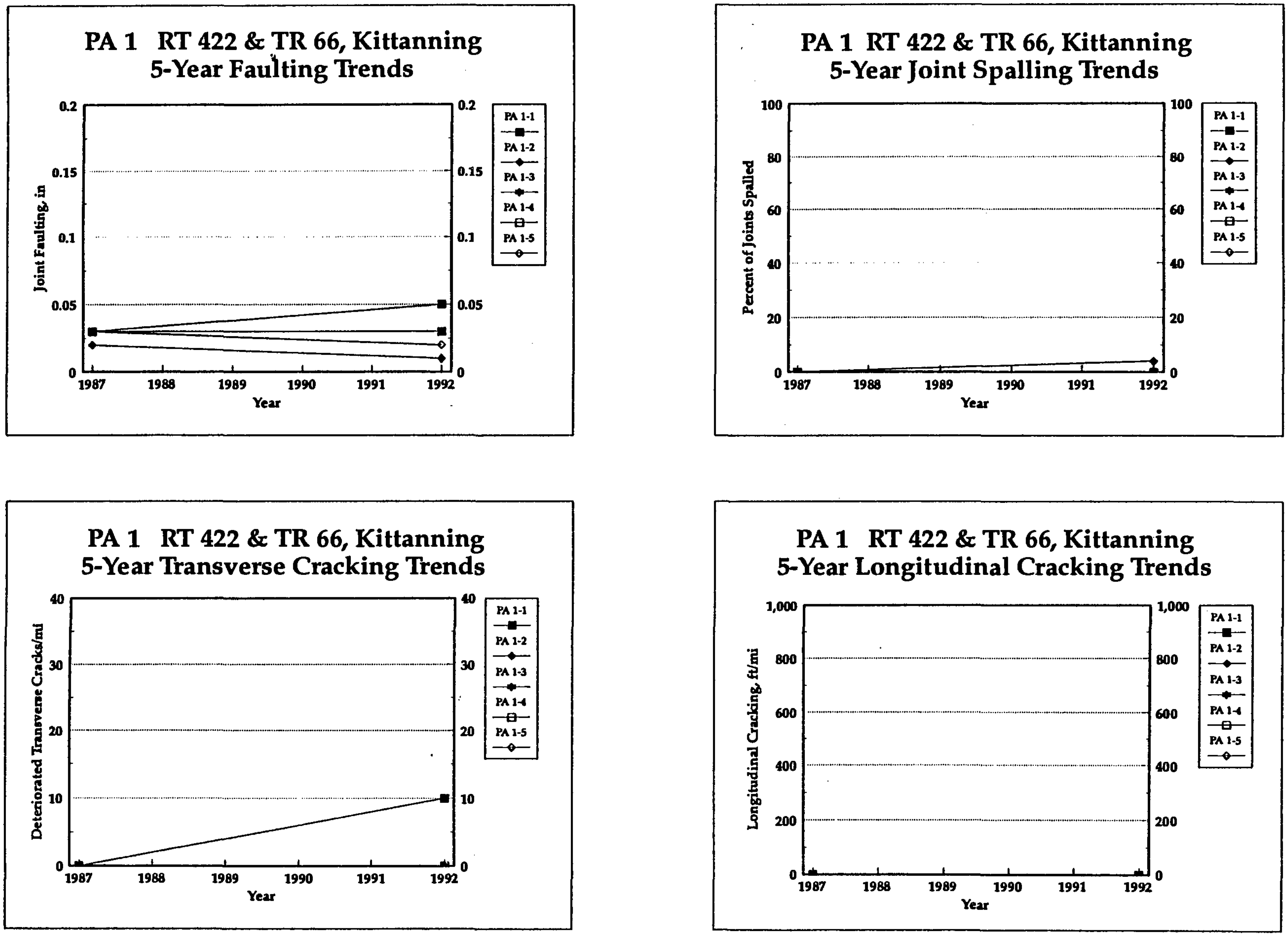


Figure 51. 5-year performance trends for PA 1 sections.

Summary

The performance exhibited by this project is summarized as follows:

- All sections are in very good condition. No section has an appreciable amount of joint faulting or spalling, and only one section (PA 1-1) has any deteriorated cracks.
- The only changes in PA 1 since 1987 are the slight increase in deterioration of transverse cracks and faulting in PA 1-1.

West Virginia 1 (I-77, Charleston)

While not part of an experimental project, three sections were evaluated on the West Virginia Turnpike (I-77), south of Charleston. These sections include two 10-in (254-mm) JRCP sections with 40-ft (12.2-m) joint spacing (WV 1-1, and WV 1-2), built in the early and mid-1980's and containing PCC shoulders. The third section is a 10-in (254-mm) JPCP with 15-ft (4.6-m) joint spacing and a widened outside lane (WV 1-3). WV 1-3 was added as a truck climbing lane adjacent to an existing 60-ft (18.3-m) JRCP. It was tied to the existing pavement using 5/8-in (16-mm) hook bolts at 30-in (762-mm) centers. All sections contain 1.25-in (32-mm) epoxy-coated dowel bars. Table 53 shows summarizes the key design variables for these sections.

Traffic Loadings

These sections are subjected to heavy traffic loadings, with the traffic patterns fairly uniform over the length of the project. Each section carries about 19,000 vehicles per day, including 23 percent heavy trucks. Because these sections were constructed in different years, the accumulated number of ESAL applications (through 1992) for each section differs. WV 1-1 has sustained 6.5 million, WV 1-2 has accumulated 8.9 million, and WV 1-3 has sustained 3.7 million.

Performance Data

The performance data for these sections are summarized in table 53. Figure 52 presents that data in graphical format. The oldest section (WV 1-2), shows the most faulting of the three sections, although the level of faulting is not that substantial. WV 1-1 shows the most transverse slab cracking, with deteriorated transverse cracks found on nearly one-half of the slabs in the survey section. The small percentage of reinforcing steel (0.1 percent) evidently was unable to hold the transverse cracks tight and prevent them from deteriorating. However, the reason why the transverse cracks have not deteriorated as much on WV 1-2 (which is older than 1-1) is not clear.

The JPCP section (1-3) is showing significant levels of faulting and cracking, although it was only 3 years old at the time of survey. This result is somewhat surprising for such a new pavement, particularly one that has a widened lane (15 ft [4.6 m] wide) and longitudinal edge drains. However, this lane was constructed

Table 53. Design and outer lane performance data for WV 1.

	WV 1-1	WV 1-2	WV 1-3
Pavement Type	JRCP (0.1% Steel)	JRCP (0.1% Steel)	JPCP
Year Built	1986	1981	1989
Thickness, in	10.0 (254 mm)	10.0 (254 mm)	10.0 (254 mm)
Joint Spacing, ft	40 (12.2 m)	40 (12.2 m)	15 (4.6 m)
Dowel Diameter, in	1.25 (32 mm)	1.25 (32 mm)	1.25 (32 mm)
Base Type	6-in (152-mm) AGG	6-in (152-mm) CTB (5% cement)	6-in (152-mm) AGG
Subgrade	AASHTO A-4	AASHTO A-4	AASHTO A-4
Performance Data	1992 Survey	1992 Survey	1992 Survey
Age at survey, years	6	11	3
ESAL's, millions	6.5	8.9	3.7
Faulting, in	0.02	0.06	0.04
Det. Tr. Crks/mi	58	11	10
% Slabs Cracked	—	—	3
Long. Cracks, ft/mi	53	0	0
% Joints Spalled	35	50	67
PSR	3.5	3.6	3.4
IRI, in/mi	168	142	168

WV 1-3 has edge drains and 15-ft outer lane.

1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km

separately and tied to the original pavement using 5/8-in (16-mm) hook bolts at 30-in (762-mm) centers, which may have been inadequate or improperly constructed.

The amount of medium- and high-severity joint spalling that occurred on all of the sections is somewhat surprising, particularly for the new JPCP section (WV 1-3). Nearly two-thirds of the joints on WV 1-3 were badly spalled and deteriorated, in spite of the presence of the silicone sealant (which exhibited adhesion loss throughout the section). WV 1-1 was also sealed with silicone, but still showed significant joint spalling (35 percent of the joints). Section 1-2, sealed with a hot-poured asphaltic material, displayed joint spalling at 50 percent of the joints.

Roughness and Serviceability

Roughness and serviceability data for the WV 1 sections are shown in figure 53. The roughness data indicate that section 1-2 is the smoothest riding pavement, with the two newer sections (1-1, 1-3) having the same roughness. These trends also appear in the serviceability data, with section 1-2 having a higher serviceability than the other two sections.

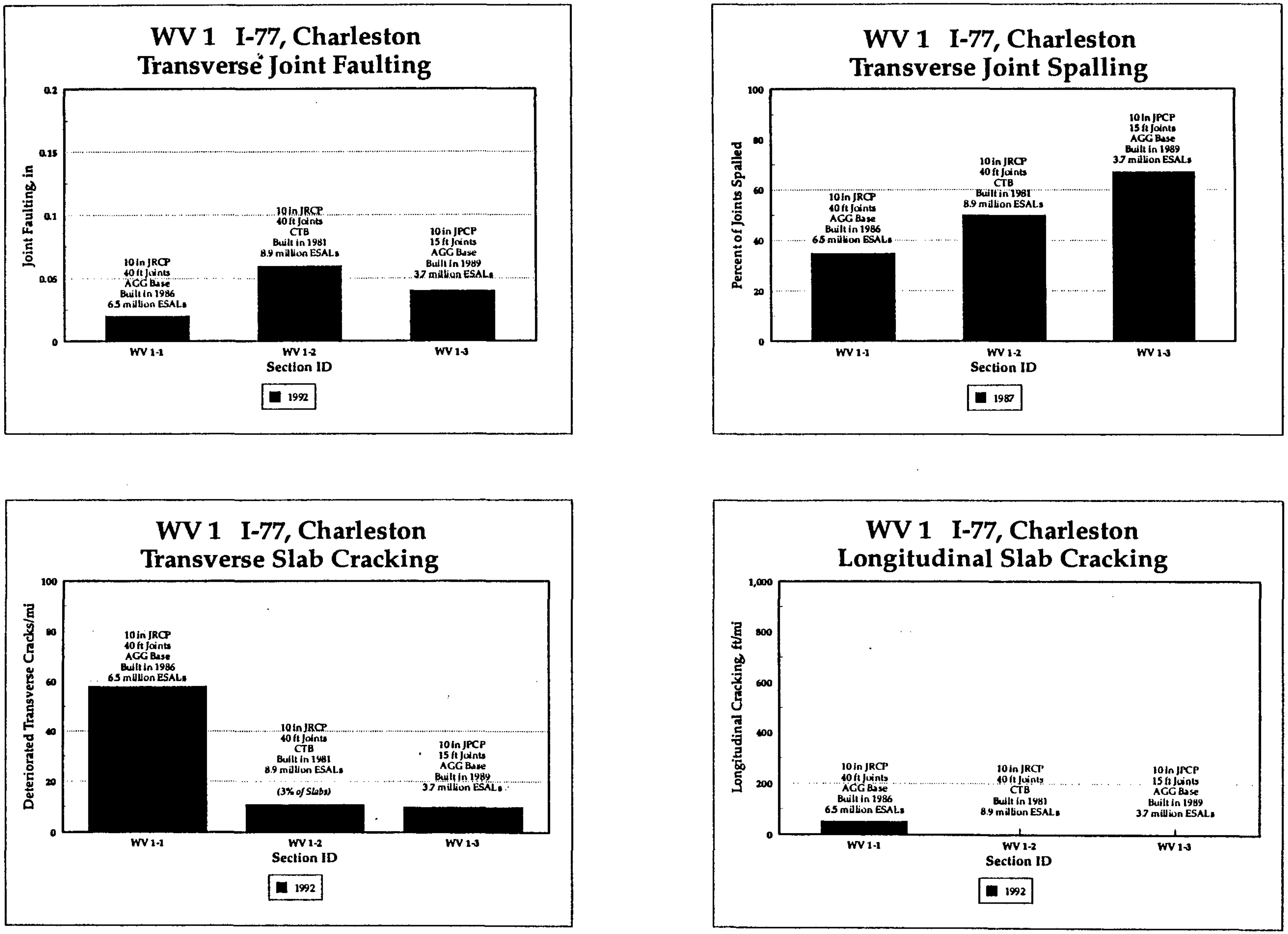
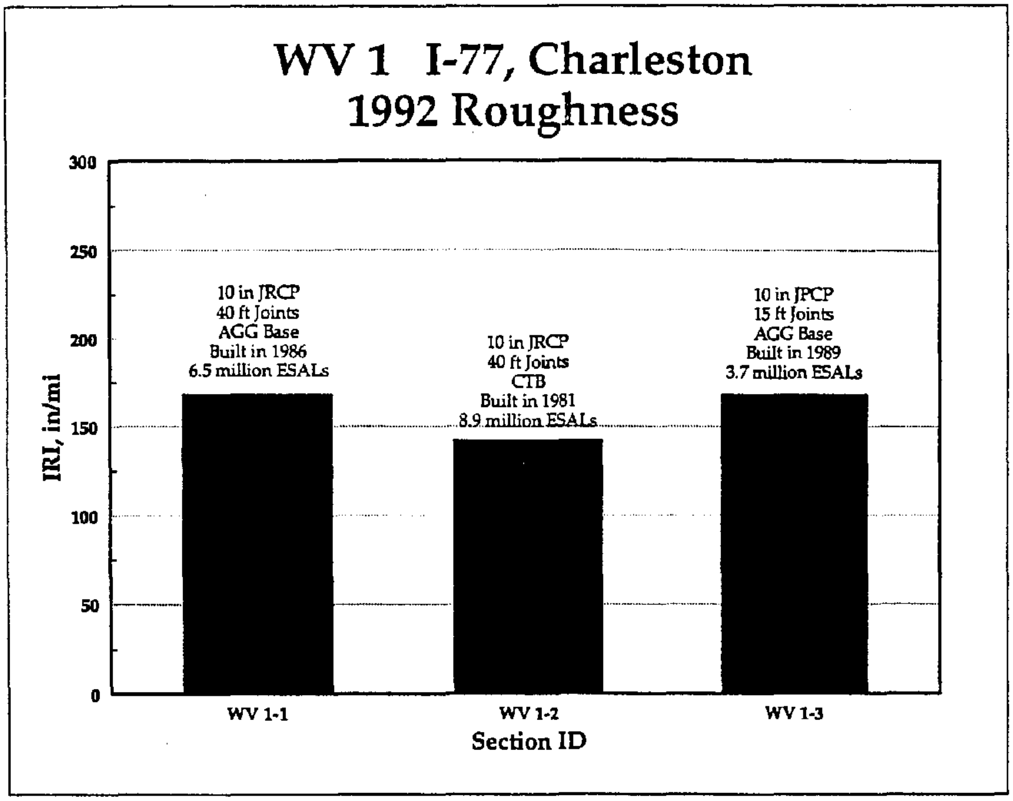


Figure 52. Performance summary for WV 1.



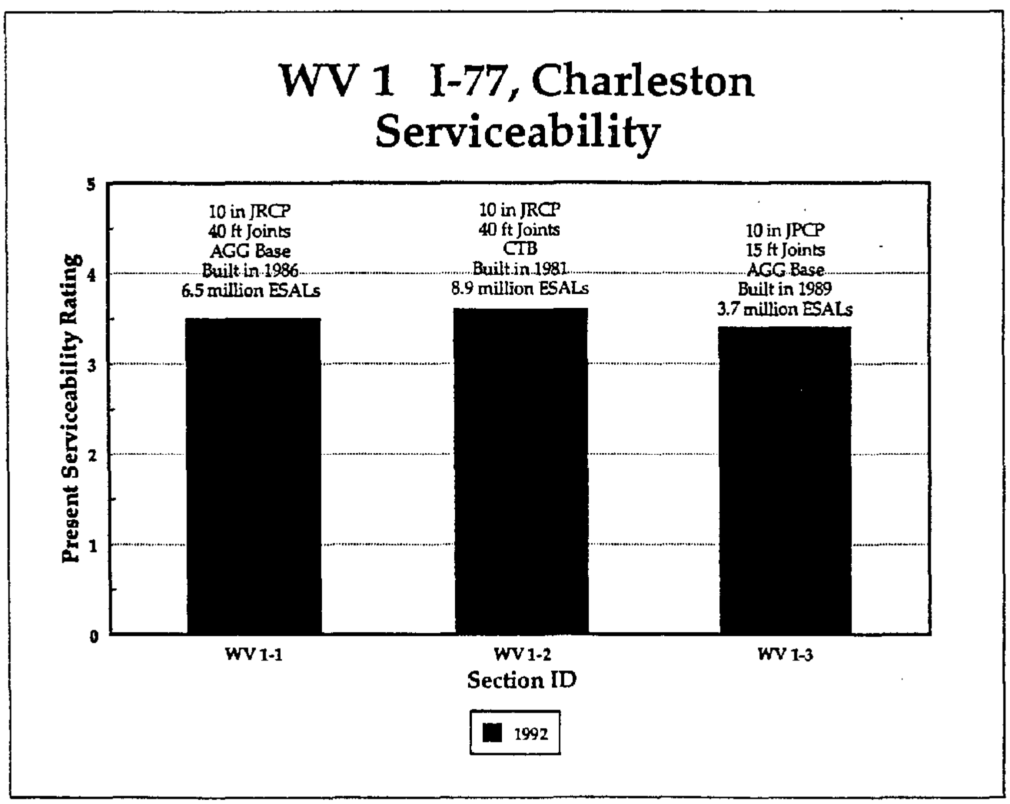


Figure 53. Roughness and serviceability for WV 1.

Summary

Three pavement sections located on I-77 near Charleston, WV, have been evaluated. The performance of these sections is summarized below:

- No section shows substantial levels of faulting at the joints. As might be expected, the oldest section (WV 1-2) shows the most faulting, although the newest section is exhibiting a significant level of faulting after only 3 years.

- The newer JRCP section shows more deteriorated cracking than the older JRCP section, which may be related to the small amount of reinforcing steel in the slab. However, because both sections contained the same amount, that does not explain why the cracks broke down on WV 1-1 and not on WV 1-2.

- The JPCP section (WV 1-3) is showing significant levels of faulting, cracking, and spalling for a pavement that is only 3 years old. The level of spalling was particularly high on this pavement, and it was noted that the silicone joint sealants had pulled away from many of the joint sidewalls

- Significant levels of joint spalling were also observed on the JRCP sections. Section WV 1-1 (sealed with silicone), exhibited joint spalling at 35 percent of the joints, while section WV 1-2 (sealed with a hot-poured asphaltic material) exhibited joint spalling at 50 percent of the joints.

- The oldest section in the project showed the least amount of roughness of the three sections. The newer sections, including the 3-year-old JPCP, showed higher roughness levels and lower serviceability ratings.

Wisconsin 1 (I-90, Stoughton)

This experimental project, constructed in 1990, is located in the westbound lanes of I-90 near Stoughton. This project was constructed by the Wisconsin Department of Transportation to evaluate the feasibility of using various types of permeable cement-treated base (PCTB) beneath concrete pavements to provide both a construction platform and positive drainage.[36]

The pavement is a 11-in (279-mm) JPCP with skewed joints spaced at 19-18-20-17-ft (5.8-5.5-6.1-5.2-m) intervals. The joints contain 1.5-in (38-mm) dowel bars and are sealed with a preformed joint sealant material. The outside traffic lane is 14-ft (4.3-m) wide.

Three different pavement sections were constructed, each containing a different amount of cement added to the PCTB: one section contained 150 lb/yd^3 (89 kg/m^3), another contained 200 lb/yd^3 (119 kg/m^3), and a third section contained 250 lb/yd^3 (148 kg/m^3). The 4-in (102-mm) permeable layer is placed over a 4-in (102-mm) dense-graded aggregate base. Table 54 shows the experimental design matrix for the WI 1 sections.

Table 54. Experimental design matrix for WI 1.

	Cement Content of PCTB		
	5.2 %	6.8 %	8.3 %
11-in (279-mm) JPCP 4-in (102-mm) PCTB 19-18-20-17-ft (5.8-5.5-6.1-5.2-m) Joints 1.5-in (38-mm) Dowels	WI 1-1	WI 1-2	WI 1-3

All sections have 14-ft (4.3-m) outer lane.

Traffic Loadings

Since being opened to traffic in 1990, this pavement has accumulated about 5.0 million ESAL applications in the outer traffic lane. The four-lane divided roadway currently carries 32,000 vehicles per day, including 25 percent heavy trucks.

Performance

The performance data collected from this project is shown in table 55 and in figure 54. As can be seen from this information, these 2-year-old sections are performing very well. Very little joint faulting is exhibited and there are no signs of any structural deterioration. However, the PSR values do not correspond well with the distress measurements.

Table 55. Summary of 1992 outer lane performance data for WI 1.

		Cement Content of PCTB		
		5.2 %	6.8 %	8.3 %
Age at Survey, years		2	2	2
ESAL's, millions		5.0	5.0	5.0
11-in (279-mm) JPCP 4-in (102-mm) PCTB 19-18-20-17-ft (5.8-5.5-6.1-5.2-m) Joints 1.5-in (38-mm) Dowels		WI 1-1	WI 1-2	WI 1-3
	Faulting, in	0.01	0.01	0.00
	Det. Tr. Crks/mi	0	0	0
	% Slabs Cracked	0	0	0
	Long. Cracks, ft/mi	0	0	0
	% Joints Spalled	0	0	0
	PSR	3.7	3.8	3.8
	IRI, in/mi	100	84	114

All sections contain a 4-in aggregate subbase and A-4 subgrade.
All sections have 14-ft (4.3-m) outer lane and AC shoulders.

1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km

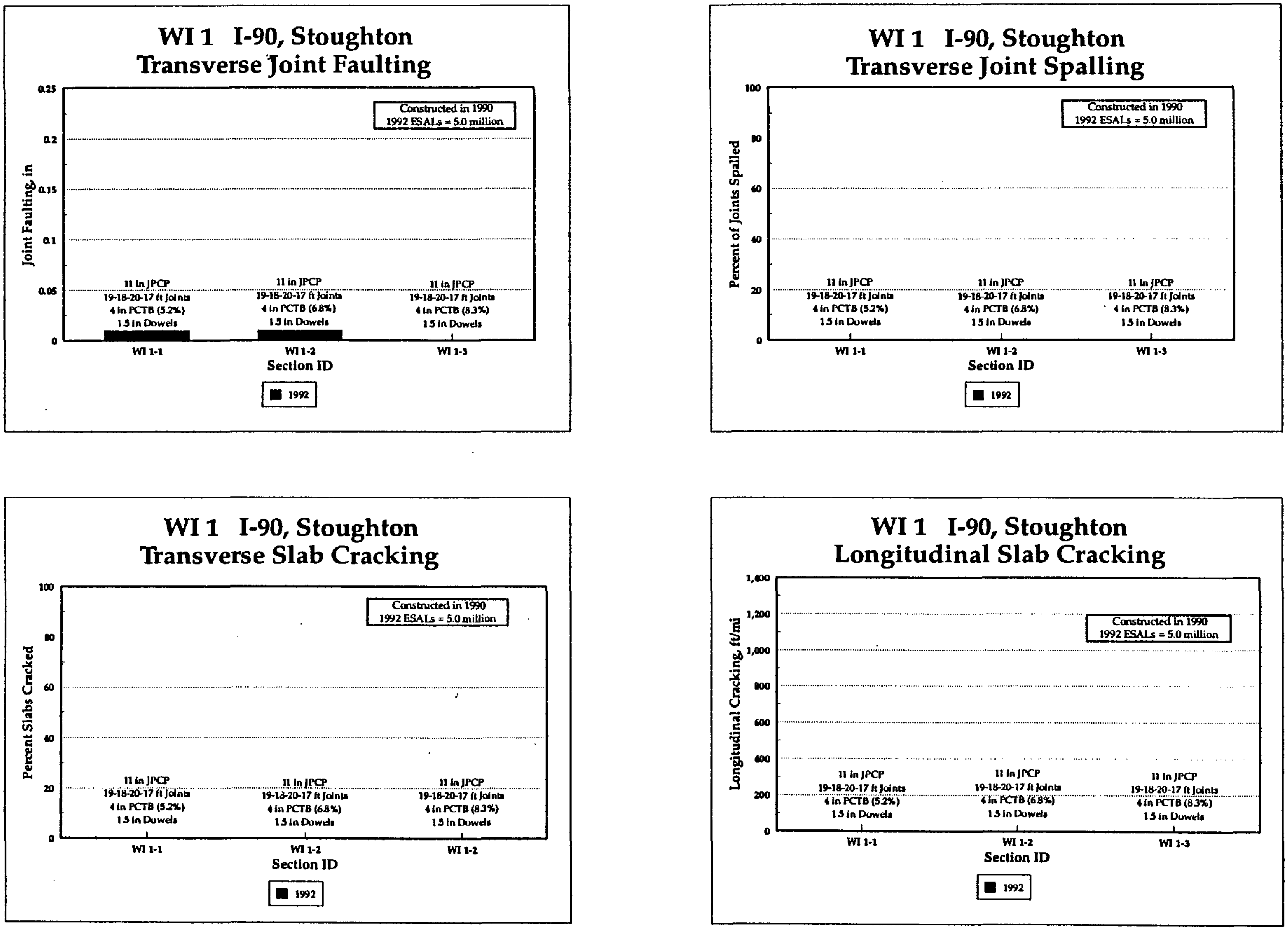


Figure 54. Performance summary for WI 1.

The usefulness of the PCTB in serving as a construction platform was one reason for constructing this experimental project. In work conducted for the Wisconsin Department of Transportation, it is reported that, prior to the placement of the concrete slab, portions of WI 1-1 and WI 1-2 (containing a lower cement content) had suffered some raveling and rutting due to the passing of haul trucks.[36] Section WI 1-3, containing the highest cement content, exhibited very little damage.

Roughness and Serviceability

Figure 55 illustrates the roughness and serviceability data for the WI 1 sections. This information shows that all sections are very smooth, with only small differences in roughness. The serviceability data also indicate that each section is providing good serviceability.

Summary

Because these sections are so new, they are exhibiting no structural deterioration; thus, no conclusions can be drawn at this time regarding their relative performance. However, data from the construction log indicate that, prior to the placement of the PCC slab, the sections with lower cement content had suffered some raveling and rutting. The section with the highest cement content exhibited little damage.

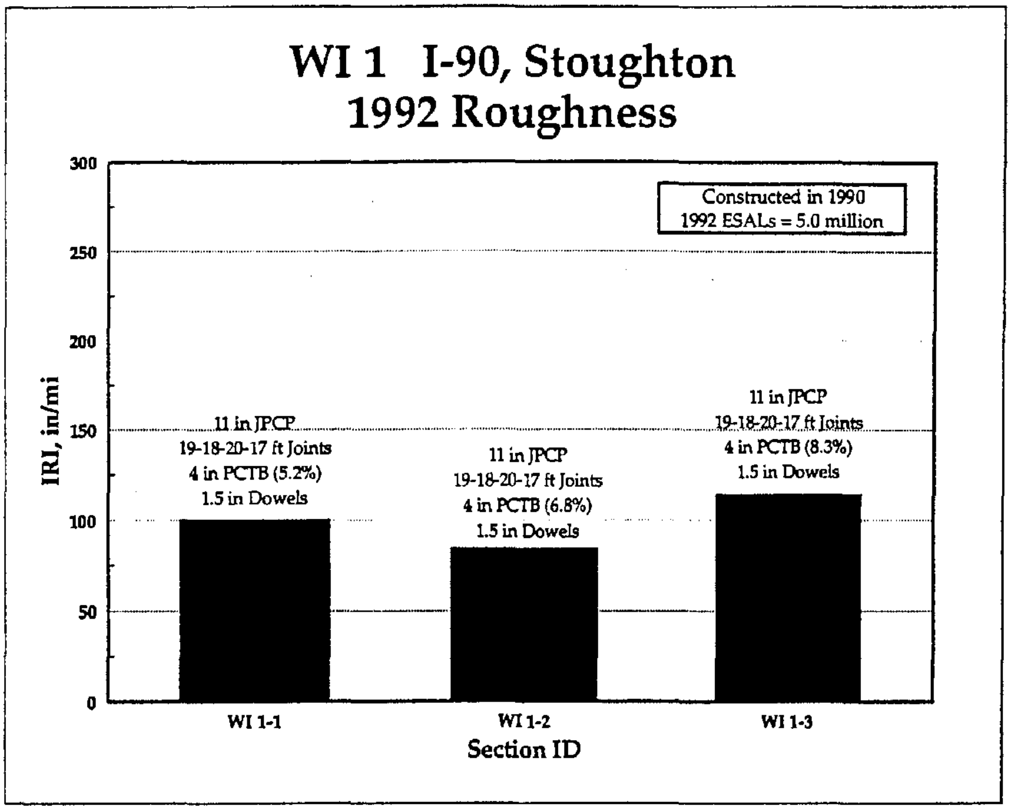
<u>Wisconsin 2 (U.S. 18/151, Mt. Horeb) and Wisconsin 7 (U.S. 18/151, Barneveld)</u>

These two projects, which are located adjacent to each other on U.S. 18/151, are presented together because of the way that they complement each other in terms of their ability to compare design features. The WI 2 sections are located near Mt. Horeb (in Dane County), while the WI 7 sections are located near Barneveld (in adjacent Iowa County).

Constructed in 1988, these pavement sections are 9-in (229-mm) JPCP with 12-13-19-18-ft (3.7-4.0-5.8-5.5-m) skewed, random joints and AC shoulders. Considering the projects together, design variables that can be evaluated include load transfer, base type (4 in [102 mm] PCTB, 4 in [102 mm] PATB, 4 in [102 mm] PAGG, and 6 in [152 mm] AGG), drainage (none, longitudinal, transverse), and joint sealant (none vs. preformed). All sections contain an aggregate subbase course with thicknesses ranging from 12 to 16 in (305 to 406 mm). WI 2 sections were constructed on an A-6 subgrade, and WI 7 sections were constructed on an A-7 subgrade. The design matrix for the WI 2 and WI 7 projects is shown in table 56.

Traffic Loadings

These pavement sections, located on a four-lane divided roadway, currently carry a two-way ADT of 9,100 vehicles per day, including 14 percent heavy trucks. Since their construction in 1988 and through 1992, they have sustained approximately 1.3 million ESAL applications in the outer traffic lane.



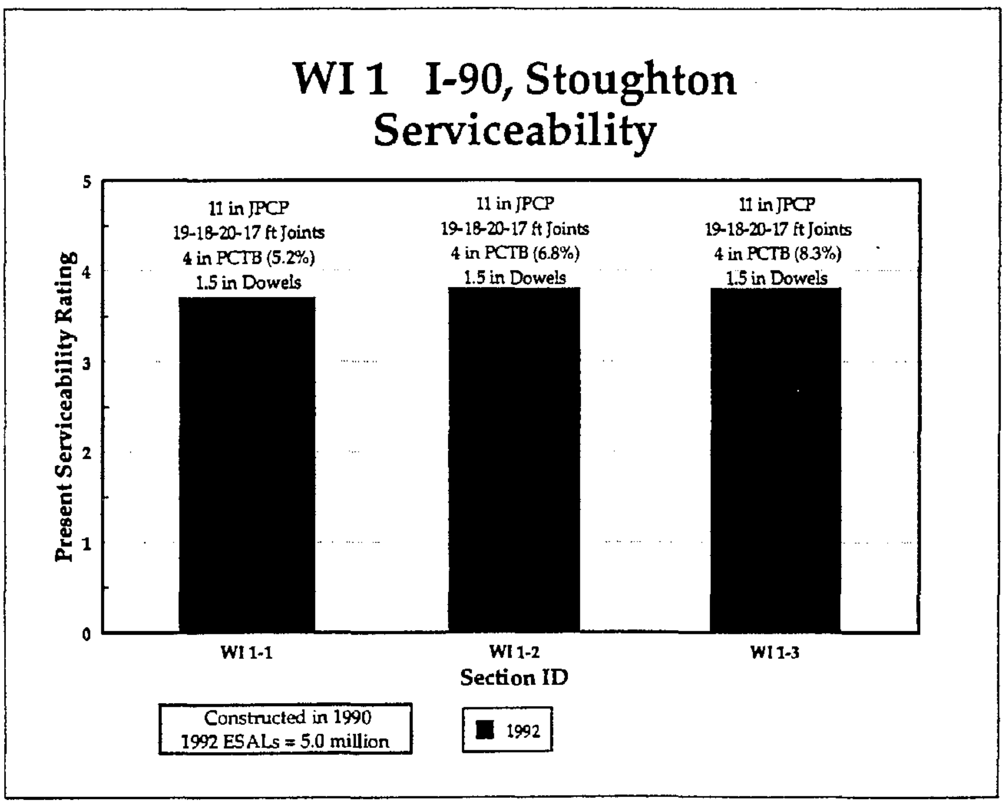


Figure 55. Roughness and serviceability for WI 1.

Table 56. Experimental design matrix for WI 2 and WI 7.

		9-in (229-mm) JPCP 12-13-19-18-ft (3.7-4.0-5.8-5.5-m) Joints				
		Longitudinal Drains			Trans. Drains	No Drains
		4-in (102-mm) PCTB (6-8%)	4-in (102-mm) PATB (2%)	4-in (102-mm) PAGG	6-in (152-mm) AGG	6-in (152-mm) AGG
1.25-in (32-mm) Epoxy-Coated Dowels	Preformed Seals					2-5
	No Seal	2-1	2-2	2-3	7-10	2-4
No Dowels	Preformed Seals	7-3	7-5	7-1		7-8
	No Seal	7-4	7-6	7-2	7-7	7-9

WI 2-4, 2-5, 7-7, 7-8, 7-9, and 7-10 have 12-in (305-mm) aggregate subbase.
Other sections have 16-in (406-mm) aggregate subbase.
All sections have 14-ft (4.3-m) outer lane and AC shoulders.
WI 2 sections constructed on A-6 subgrade; WI 7 sections constructed on A-7 subgrade.
WI 2-2 = SHRP 556355, WI 7-4 = SHRP 556353, WI 7-6 = SHRP 556354, WI 7-10 = SHRP 556352.

Performance

The performance data for the WI 2 and WI 7 experimental sections are presented in table 57 and in figure 56. An examination of this information indicates that these sections are performing very well after 4 years of service. Because of the small amount of distress that has developed, it is difficult to draw definitive conclusions regarding the relative performance of the section. For example, the faulting on the doweled sections is consistently lower than that of the nondoweled sections, although the difference is not significant. As another example, sections with preformed joint seals have less spalling than nonsealed sections, although again the difference is not significant. Finally, the PATB sections show less faulting than the PCTB or the PAGG, but this difference is not significant.

Some subtle indications show that drainage has an effect on reducing faulting. For example, the nondoweled sections with permeable bases show less faulting than the nondrained section constructed on a dense-graded aggregate base. The drained section on dense-graded aggregate base also exhibits less faulting than the nondrained sections with dense-graded aggregate base.

Although some sections display slab cracking, the amount exhibited is not significant and is not detracting from the performance of the pavement.

Table 57. Summary of 1992 outer lane performance data for WI 2 and WI 7.

			9-in (229-mm) JPCP 12-13-19-18-ft (3.7-4.0-5.8-5.5-m) Skewed Joints					Average
			Longitudinal Drains			Trans. Drains	No Drains	
			4-in PCTB (6-8%)	4-in PATB (2%)	4-in PAGG	6-in AGG	6-in AGG	
			1992 Survey	1992 Survey	1992 Survey	1992 Survey	1992 Survey	1992 Survey
Age at Survey, years			4	4	4	4	4	4
ESAL's, millions			1.3	1.3	1.3	1.3	1.3	1.3
1.25-in (32-mm) Epoxy-Coated Dowels	Preformed Seals						2-5	
		Faulting, in					0.01	0.01
		Det. Tr. Crks/mi					0	0
		% Slabs Cracked					0	0
		Long. Cracks, ft/mi					0	0
		% Joints Spalled					0	0
		PSR					3.9	3.9
		IRI, in/mi					122	122
	No Seal		2-1	2-2	2-3	7-10	2-4	
		Faulting, in	0.01	0.01	0.01	0.01	0.01	0.01
		Det. Tr. Crks/mi	0	0	0	0	0	0
		% Slabs Cracked	0	0	0	0	0	0
		Long. Cracks, ft/mi	0	0	0	0	0	0
		% Joints Spalled	5	3	3	3	6	4
		PSR	4.1	4.0	4.0	4.2	4.0	4.1
		IRI, in/mi	122	106	112	113	121	115
No Dowels	Preformed Seals		7-3	7-5	7-1		7-8	
		Faulting, in	0.01	0.01	0.03		0.04	0.02
		Det. Tr. Crks/mi	0	0	0		0	0
		% Slabs Cracked	0	0	0		0	0
		Long. Cracks, ft/mi	326	0	0		0	0
		% Joints Spalled	0	2	6		3	3
		PSR	4.1	4.1	4.3		3.9	4.1
		IRI, in/mi	147	86	136		116	121
	No Seal		7-4	7-6	7-2	7-7	7-9	
		Faulting, in	0.04	0.02	0.03	0.02	0.04	0.03
		Det. Tr. Crks/mi	0	0	21	0	0	8
		% Slabs Cracked	0	0	6	0	0	2
		Long. Cracks, ft/mi	257	0	0	0	0	51
		% Joints Spalled	8	10	8	3	8	7
		PSR	4.0	4.0	4.5	4.3	3.9	4.1
		IRI, in/mi	147	98	120	117	114	119
Average		Faulting, in	0.02	0.01	0.02	0.02	0.02	
		Det. Tr. Crks/mi	0	0	7	0	0	
		% Slabs Cracked	0	0	2	0	0	
		Long. Cracks, ft/mi	194	0	0	0	0	
		% Joints Spalled	4	5	6	3	4	
		PSR	4.1	4.0	4.3	4.2	3.9	
		IRI, in/mi	139	97	123	115	118	

WI 2-4, 2-5, 7-7, 7-8, 7-9, and 7-10 have 12-in (305-mm) aggregate subbase.
Other sections have 16-in (406-mm) aggregate subbase.
All sections have 14-ft (4.3-m) outer lane.
WI 2 sections constructed on A-6 subgrade; WI 7 sections constructed on A-7 subgrade.
WI 2-2 = SHRP 556355, WI 7-4 = SHRP 556353, WI 7-6 = SHRP 556354, WI 7-10 = SHRP 556352.

1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.6 km

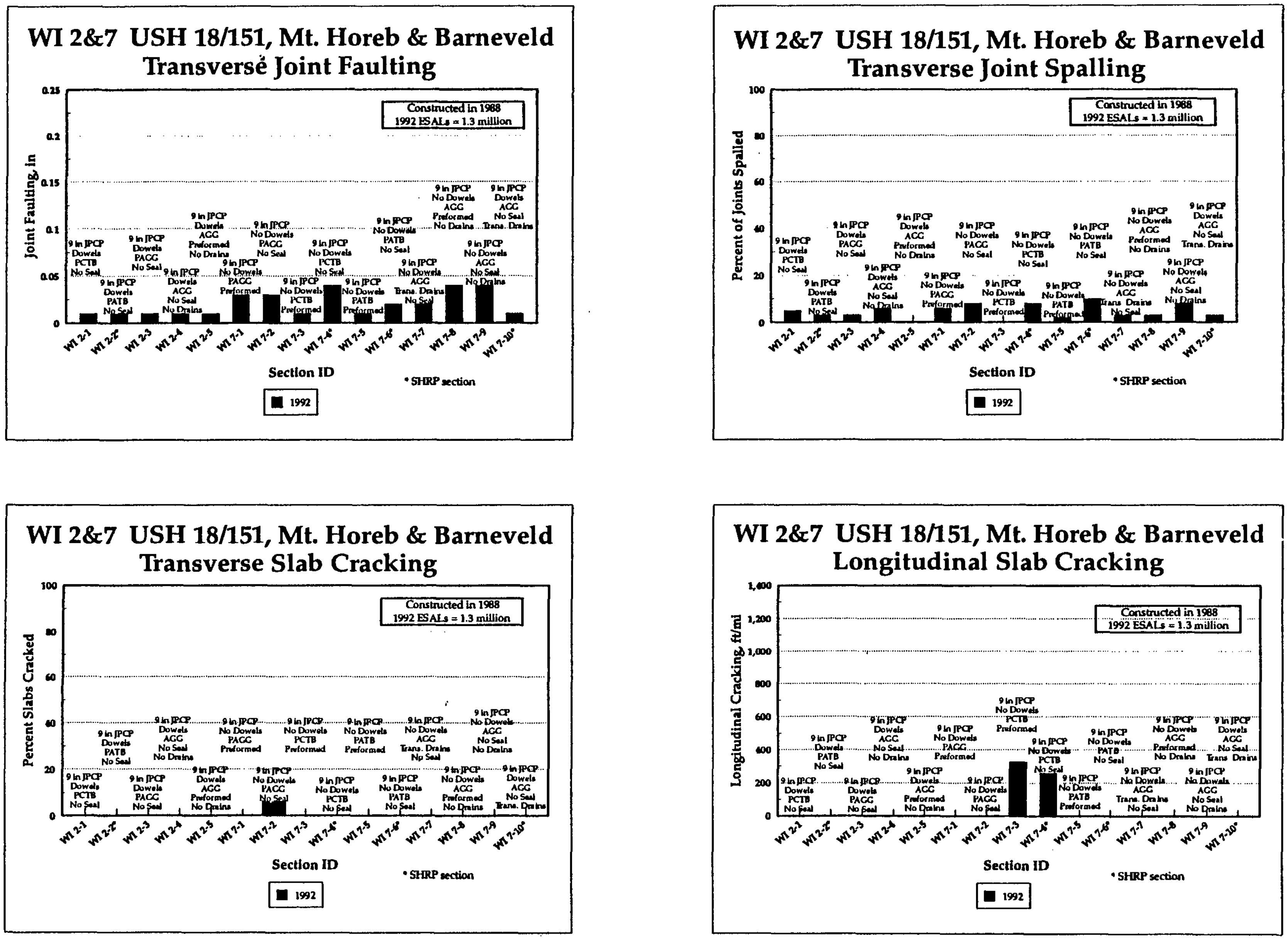


Figure 56. Performance summary for WI 2 and WI 7.

Roughness and Serviceability

Figure 57 presents the roughness and serviceability data for the WI 2 and WI 7 sections. The roughness data for most of these sections are clustered relatively close together, with a few exceptions. For example, the smoothest sections on the project are WI 7-5 and WI 7-6, which are both constructed on PATB. This result is in contrast with sections 7-3 and 7-4, which are the two roughest sections on the project and are constructed on PCTB. By also considering section 7-1 (constructed on PAGG), these data may suggest that the PATB can be easily constructed and can provide a stable working platform for paving operations. Often, PAGG can be difficult to construct and may be unstable under construction traffic, exhibiting rutting or shoving. For this reason, some states prohibit hauling on permeable bases.

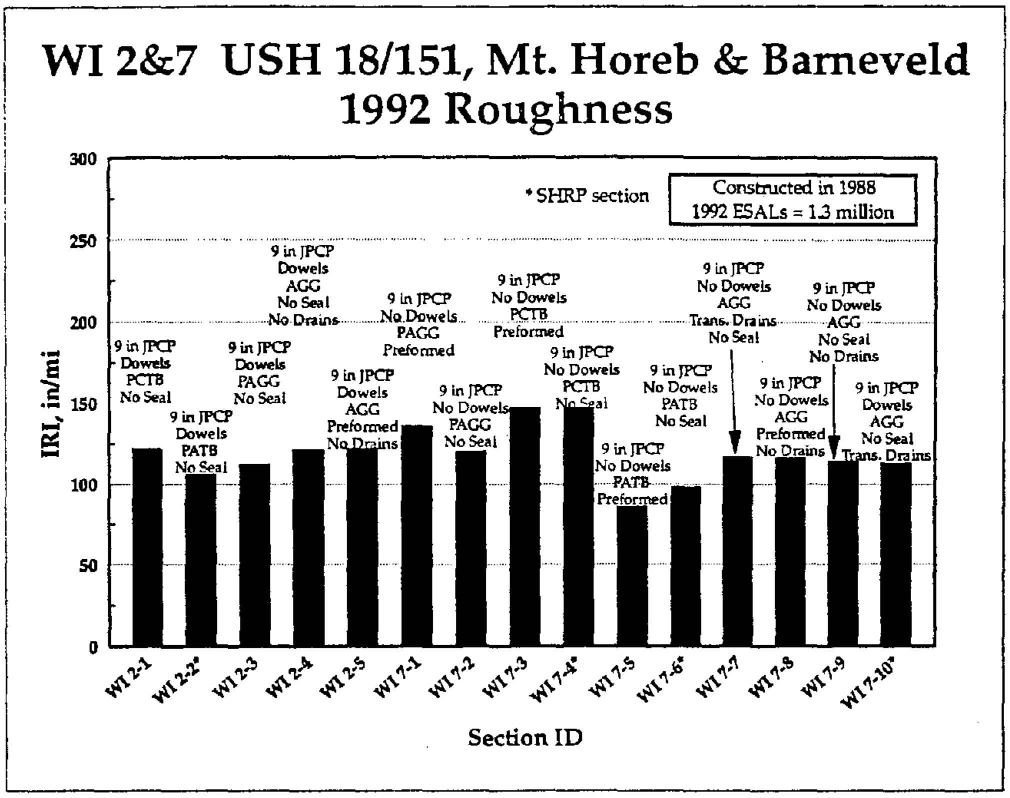
For the most part, the trends shown by the serviceability data mirror that of the roughness data. The serviceability values all indicate that the sections are providing good rideability.

Summary

Because of the low levels of distress exhibited by the WI 2 and WI 7 sections, it is difficult to draw reliable conclusions from the data. However, the data do indicate some subtle performance indications that will be worth tracking in future evaluations:

- The faulting on the doweled sections is consistently lower than that of the nondoweled sections.
- The drained sections display less faulting than the nondrained sections.
- Pavement sections with preformed joint seals have less spalling than nonsealed sections.
- The PATB sections show less faulting than the PCTB or the PAGG.
- The PATB sections are smoother than any of the other sections, perhaps reflecting the efficacy of that base type in providing a smooth and stable working platform for construction activities.

However, at this time, each observation must be tempered with the understanding that the levels of distress that have occurred on all sections are not substantial. It will be interesting to determine how these initial observations hold up after the sections have been subjected to additional traffic loadings.



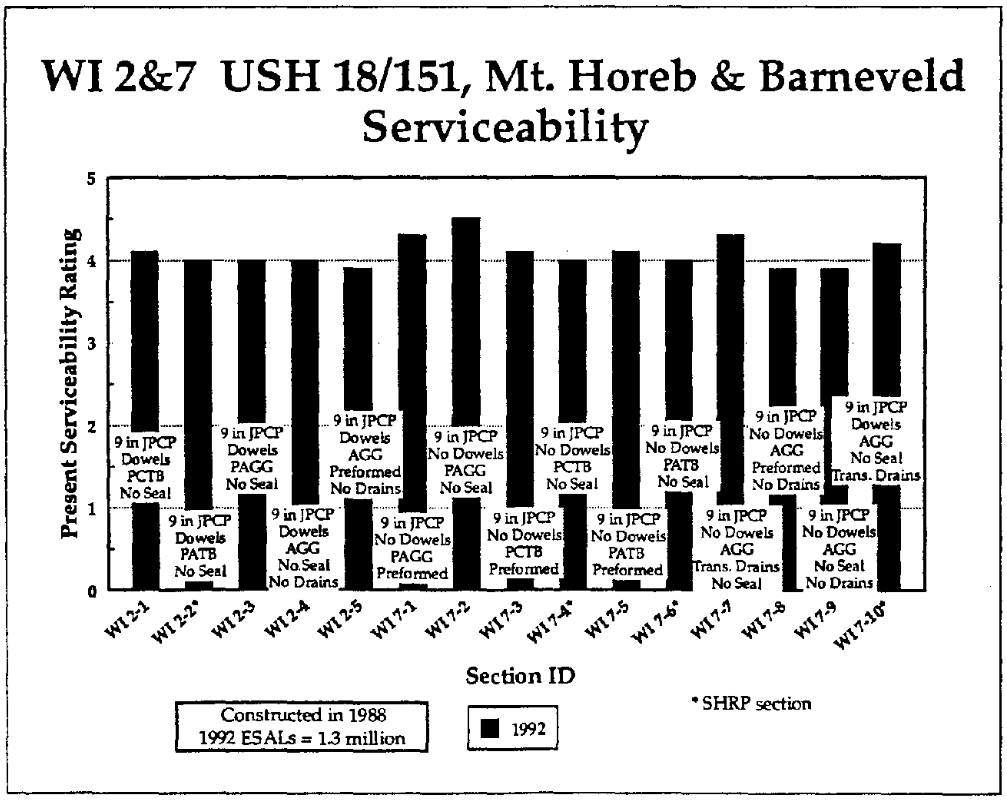


Figure 57. Roughness and serviceability for WI 2 and WI 7.

Wisconsin 3 (STH 14, Middleton)

This experimental project, a two-lane highway, is located on STH 14 near Middleton. Built in 1988, the project consists of three, nondoweled 8-in (203-mm) JPCP sections whose joints are spaced at 12-13-19-18-ft (3.7-4.0-5.8-5.5-m) intervals. Two design features are evaluated under this project: drainage (longitudinal fin drain vs. no drain) and base type (3.5-in [89-mm] permeable asphalt-treated base vs. 6-in [152-mm] dense-graded aggregate base). The experimental design matrix for this project is given in table 58.

Table 58. Experimental design matrix for WI 3.

	8-in (203-mm) JPCP 12-13-19-18-ft (3.7-4.0-5.8-5.5-m) Skewed Jts No Dowels	
	3.5-in (89-mm) PATB (2%)	6-in (152-mm) AGG
Longitudinal Fin Drain	WI 3-1	WI 3-2
No Drain		WI 3-3

WI 3-1 has 6-in (152-mm) aggregate subbase.
Other sections have no subbase.
All sections have AC shoulders and A-3 subgrade.

Traffic Loadings

As of 1992, the two-lane roadway has sustained 1.2 million ESAL applications since its construction in 1988. It currently is subjected to a two-way ADT of 8,700 vehicles per day, including 12 percent heavy trucks.

Performance

The performance data for this project are summarized in table 59. This information is portrayed in figure 58 in graphical format. With the exception of joint faulting, these sections are relatively free of distress.

The performance data show that the section constructed on the PATB (WI 3-1) is exhibiting substantially less faulting than the adjacent section constructed on the dense-graded aggregate base (WI 3-2). Thus, the section with the PATB is much more effective in removing water from the pavement structure than the dense-graded section, even though both sections contained longitudinal fin drains. For section 3-2, this could suggest that either the water is not reaching the fin drain or that the fin drain is not effective in removing water. Because this substantial difference in performance is apparent after only 4 years of service, it will be interesting to observe what future trends are exhibited by these sections.

Table 59. Summary of 1992 performance data for WI 3.

		8-in (229-mm) JPCP 12-13-19-18-ft (3.7-4.0-5.8-5.5-m) Skewed Jts No Dowels	
		3.5-in (89-mm) PATB (2%)	6-in (152-mm) AGG
		1992 Survey	1992 Survey
Age at Survey, years		4	4
ESAL's, millions		1.2	1.2
Longitudinal Fin Drain		WI 3-1	WI 3-2
	Faulting, in	0.03	0.13
	Det. Tr. Crks/mi	0	0
	% Slabs Cracked	0	0
	Long. Cracks, ft/mi	0	0
	% Joints Spalled	2	3
	PSR	4.0	3.6
	IRI, in/mi	109	151
No Drain			WI 3-3
	Faulting, in		0.15
	Det. Tr. Crks/mi		0
	% Slabs Cracked		0
	Long. Cracks, ft/mi		0
	% Joints Spalled		2
	PSR		3.5
	IRI, in/mi		147

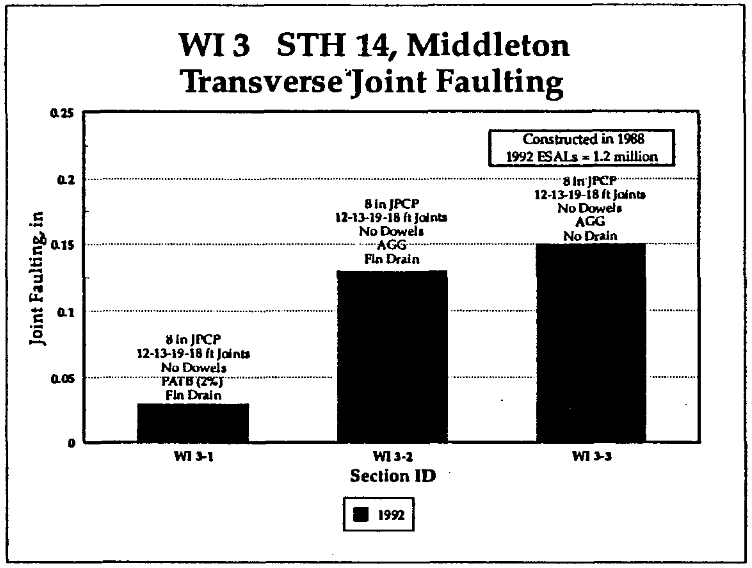
WI 3-1 has 6-in (152-mm) aggregate subbase.
Other sections have no subbase.
All sections have AC shoulder and A-3 subgrade.

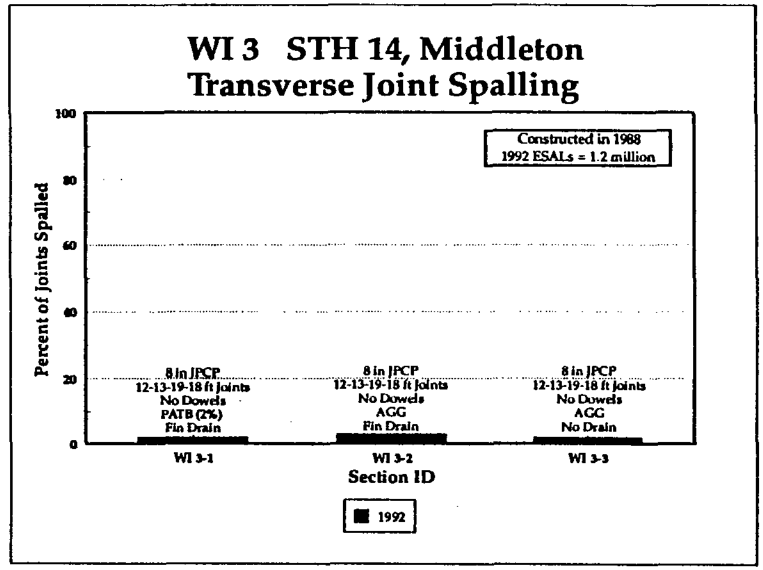
1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km

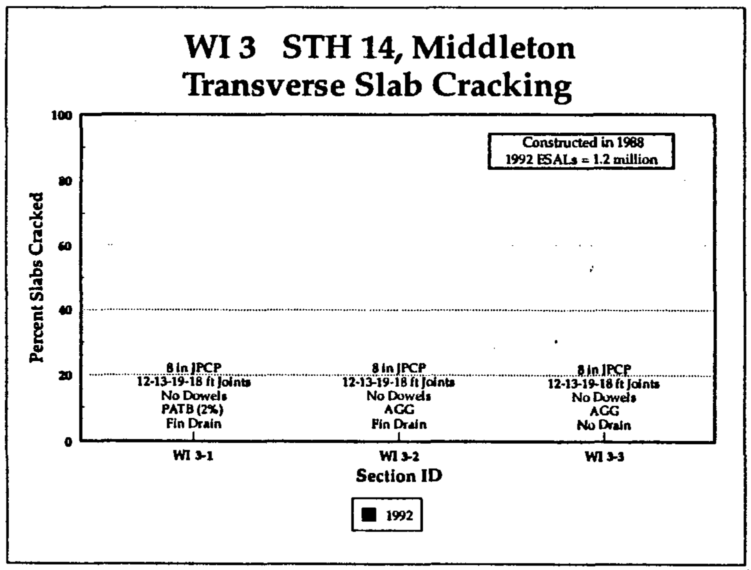
In comparing the performance of WI 3-2 and WI 3-3, the benefits of the longitudinal fin drain are not apparent. While WI 3-2 (which contains the fin drain) has slightly less faulting than WI 3-3, the overall difference in faulting is not significant. The fact that both section WI 3-2 and WI 3-3 have attained critical levels of faulting (greater than 0.12 in [3 mm]) after only 4 years indicates that neither design is effective in reducing faulting. A more positive means of load transfer (dowel bars) and drainage (permeable base) are required for the given climate and level of traffic.

Roughness and Serviceability

The roughness and serviceability data for the WI 3 sections are shown in figure 59. This information shows that, as expected, the section constructed on the permeable base is much smoother and has a higher serviceability rating than the other two sections. The critical levels of faulting exhibited by WI 3-2 and 3-3 no doubt contribute to the roughness recorded on those sections.







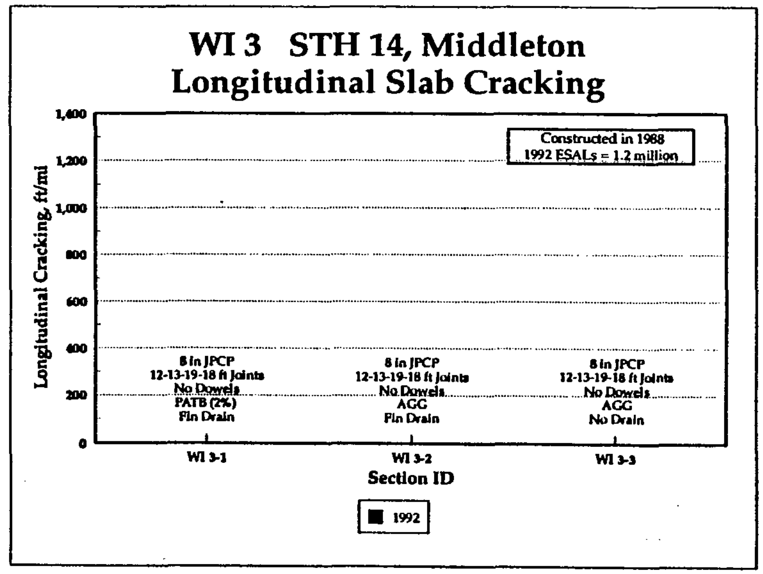
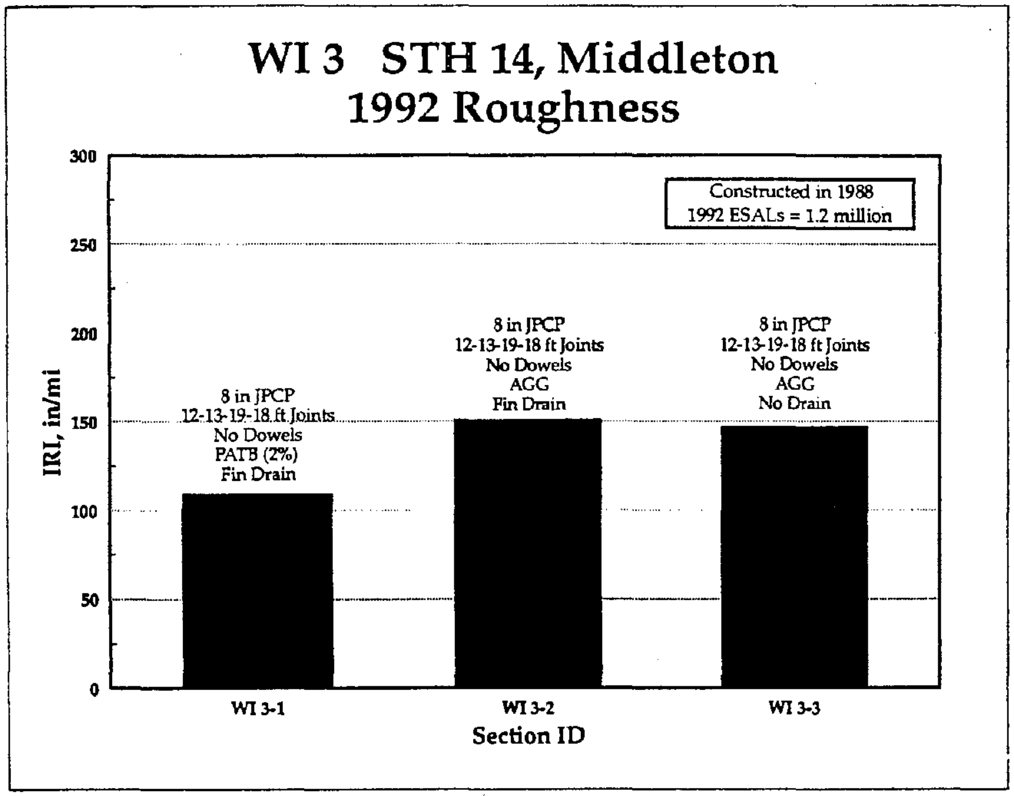


Figure 58. Performance summary for WI 3.



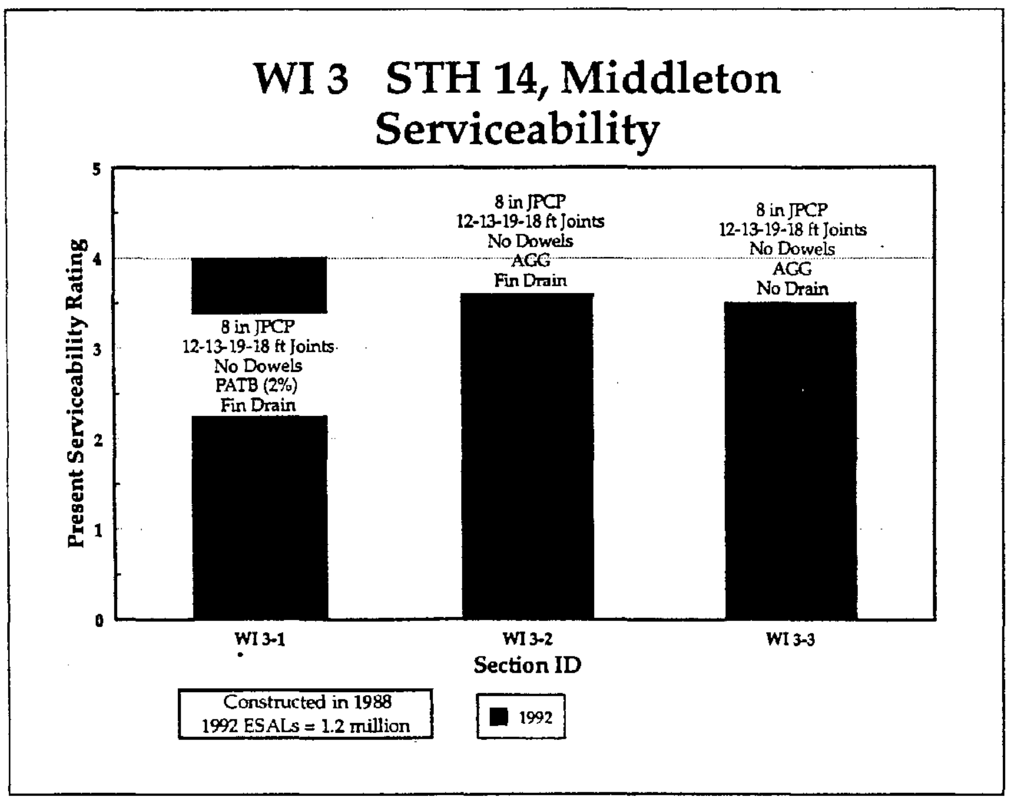


Figure 59. Roughness and serviceability for WI 3.

Summary

The performance of the experimental concrete pavement sections constructed on STH 14 near Middleton is summarized below:

- The pavement section constructed on the PATB shows much less faulting than the corresponding section placed on a dense-graded base, even though both sections contained a longitudinal fin drain. This result might suggest that either the water cannot reach the fin drain or that the fin drain is not functioning effectively.

- No apparent difference in performance is apparent between the drained and nondrained sections placed on aggregate base. Although the faulting on the drained section is slightly lower, the faulting on both sections has reached critical levels that significantly detract from the rideability of the pavement.

- The section placed on PATB is also much smoother than either of the other two sections. The critical levels of faulting on the other two sections is no doubt contributing to the roughness of those pavements.

<u>Wisconsin 4 (STH 164, Waukesha)</u>

This project consists of five sections in the northbound lanes and one section in the southbound lanes of STH 164 in Waukesha. Constructed in 1988, the pavement design is a 9-in (229-mm) JPCP with tied PCC shoulders placed on a 6-in (152-mm) dense-graded aggregate base. The joints are spaced at 20-ft (6.1-m) intervals and do not contain dowel bars.

As seen in table 60, design variables include transverse joint drains, longitudinal edge drains, and sealed joints. Unfortunately, five of the experimental sections (4-1 through 4-5) are located in the center lane of a three-lane roadway in an urban location. Because of the problems in obtaining traffic control for the survey of those urban sections, only one section (4-6, in the southbound direction) was surveyed using manual methods. However, distress data from the automated distress surveys are available for the other five sections, along with profile measurements and serviceability ratings.

Traffic Loadings

As previously mentioned, these sections are located in an urban setting. Sections 4-1 through 4-5 are in the center lanes of a six-lane roadway, and section 4-6 is in the outer traffic lane of a four-lane roadway. All sections are subjected to a two-way ADT of 22,800 vehicles per day, including 7 percent trucks. Through the year 1992, sections 4-1 through 4-5 have sustained an estimated 0.5 million ESAL applications in the center lane, whereas section 4-6 has sustained an estimated 1.5 million ESAL's in the outer lane.

Table 60. Experimental design matrix for WI 4.

		9-in (229-mm) JPCP 20-ft (6.1-m) Skewed Joints 6-in (152-mm) AGG No Dowels	
		Longitudinal Edge Drain[2]	**No Edge Drain**
Transverse Joint Drain[1]	**Joint Seal**	WI 4-3	
	No Seal	WI 4-4	
No Transverse Joint Drain	**Joint Seal**	WI 4-2	WI 4-1
	No Seal	WI 4-6	WI 4-5

[1] 2-in (25-mm) fabric-wrapped, perforated PVC pipe and 3-in (76-mm) aggregate-filled trench discharging to the edge drains.
[2] 4-in (102-mm) fabric-wrapped, perforated PVC pipe and 12- by 12-in (305- by 305-mm) aggregate-filled trench discharging to manhole inlets.

Performance

The available performance data for the WI 4 sections are provided in table 61 and displayed graphically in figure 60. The age of these sections (4 years) and the absence of faulting data for all but one section make the interpretation of the data difficult. The amount of faulting exhibited by WI 4-6 (0.08 in [2 mm]) is considered somewhat high for a pavement that is only 4 years old. When the faulting of section WI 4-6 is considered in conjunction with the faulting of WI 3-2, it appears to suggest that the presence of edge drains by themselves are not enough to ensure the reduction of faulting for nondoweled slabs.

The performance data also indicate that sections with sealed joints are showing less joint spalling than nonsealed sections. Two nonsealed sections in particular, WI 4-5 and 4-6, are exhibiting significant levels of spalling for a 4-year-old pavement.

The levels of transverse slab cracking exhibited by some of the sections is somewhat surprising, since the pavement has only been in service for 4 years. WI 4-3 and 4-5 are showing over 8 percent of the slabs cracked, and WI 4-1 and 4-2 show 3 and 4 percent, respectively. There do not appear to be any trends in the development of the slab cracking.

Roughness and Serviceability

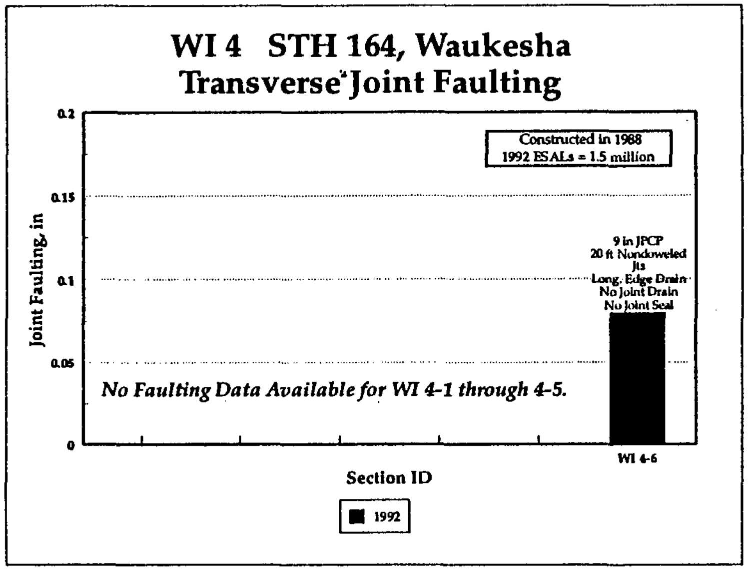
Figure 61 depicts the roughness and serviceability data collected for the WI 4 sections. For the most part, the roughness of these sections are clustered together, although there is a significant range of roughness values. The smoothest section is noted to be WI 4-4, while the roughest is WI 4-1. These two sections are quite different in design, as WI 4-4 contains both transverse and longitudinal drains but no joint sealant, whereas WI 4-1 contains joint sealant but no drainage features. The serviceability data indicate no significant differences in the rideability.

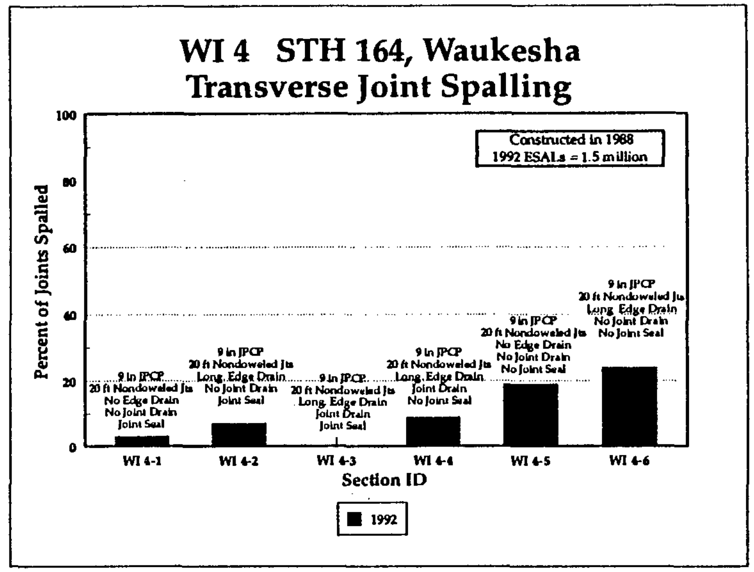
Table 61. Experimental design matrix for WI 4.

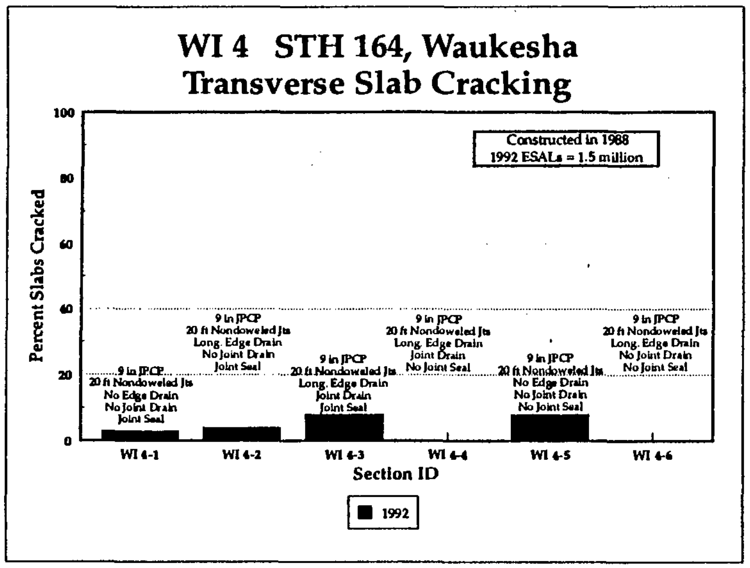
			9-in (229-mm) JPCP 20-ft (6.1-m) Joints 6-in (152-mm) AGG No Dowels	
			Longitudinal Edge Drain	**No Edge Drain**
Age at Survey, years			4	4
ESAL's, millions			0.5	0.5
Transverse Joint Drain	**Joint Seal**		WI 4-3	
		Faulting, in	—	
		Det. Tr. Crks/mi	20	
		% Slabs Cracked	8	
		Long. Cracks, ft/mi	0	
		% Joints Spalled	0	
		PSR	3.8	
		IRI, in/mi	121	
	No Seal		WI 4-4	
		Faulting, in	—	
		Det. Tr. Crks/mi	0	
		% Slabs Cracked	0	
		Long. Cracks, ft/mi	0	
		% Joints Spalled	9	
		PSR	3.7	
		IRI, in/mi	109	
No Joint Drain	**Joint Seal**		WI 4-2	WI 4-1
		Faulting, in	—	—
		Det. Tr. Crks/mi	9	9
		% Slabs Cracked	4	3
		Long. Cracks, ft/mi	0	0
		% Joints Spalled	7	3
		PSR	3.7	3.8
		IRI, in/mi	129	135
	No Seal		WI 4-6*	WI 4-5
		Faulting, in	0.08	—
		Det. Tr. Crks/mi	0	21
		% Slabs Cracked	0	8
		Long. Cracks, ft/mi	0	0
		% Joints Spalled	24	19
		PSR	3.8	3.8
		IRI, in/mi	125	120

* WI 4-6 has sustained an estimated 1.5 million ESAL's.
* WI 4-6 is from manual survey; others are from PASCO data.
All sections contain AC shoulders.

1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km







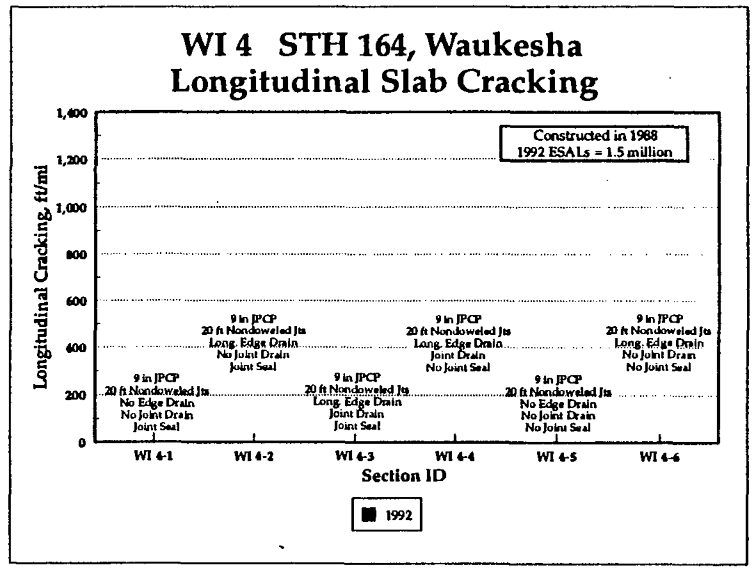
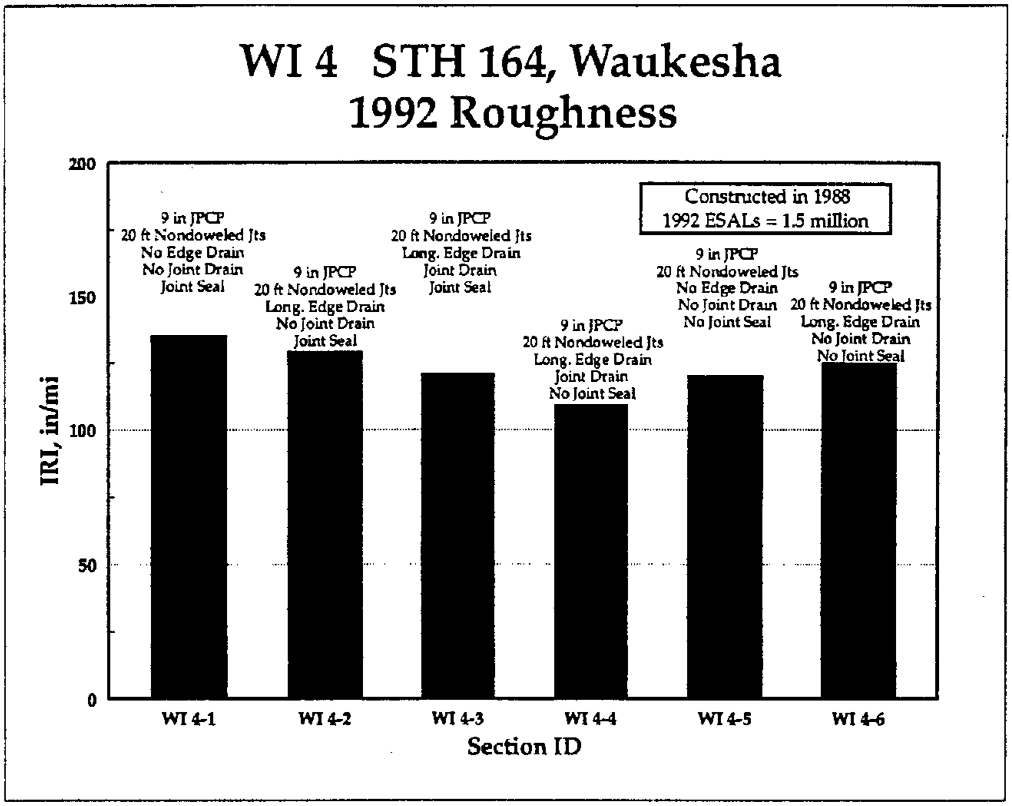


Figure 60. Performance summary for WI 4.



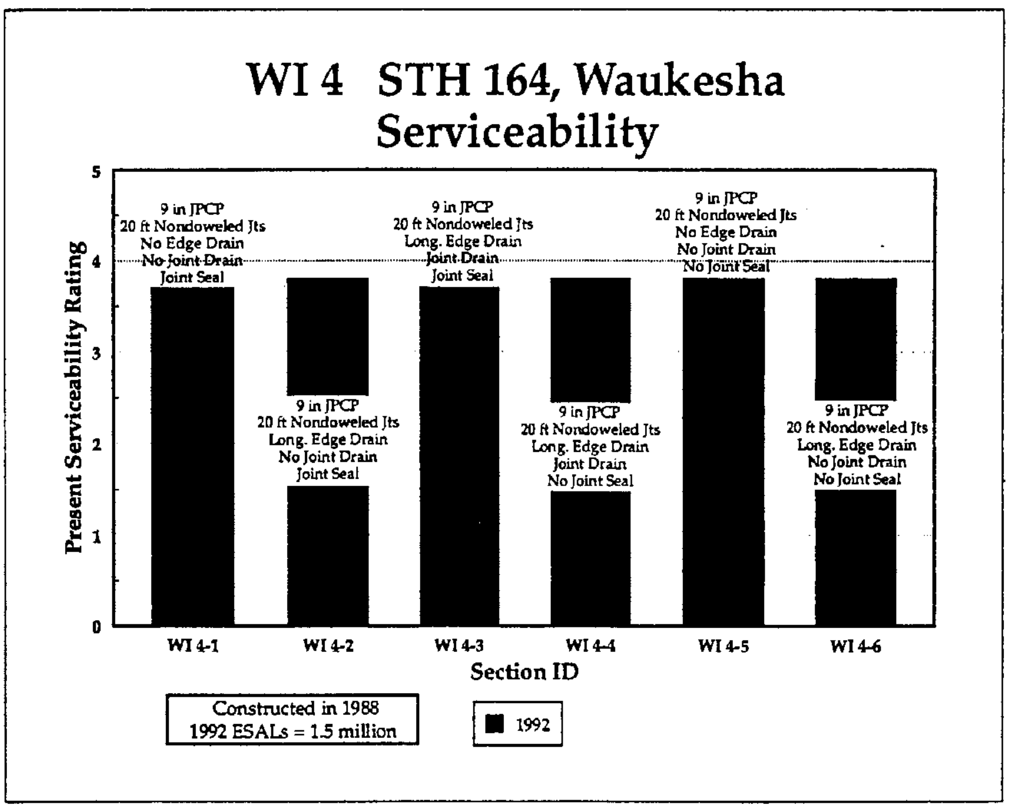


Figure 61. Roughness and serviceability for WI 4.

Summary

Because five of the six WI 4 sections are located in the center lane of a six-lane urban roadway, a manual distress survey was conducted only on one section. However, some distress data is available from the automated distress surveys so that some performance comparisons can be made. A summary of the performance of these sections is given below:

- The one section for which faulting is available (WI 4-6) exhibited a significant level of faulting after only 4 years. When the faulting of section WI 4-6 is considered in conjunction with the faulting of section WI 3-2, this may suggest that the presence of edge drains by themselves are not enough to ensure the reduction of faulting for nondoweled slabs.

- The pavement sections with sealed joints are showing less joint spalling than the nonsealed sections. Two nonsealed sections in particular, WI 4-5 and 4-6, are exhibiting high levels of spalling.

- Some sections displayed significant amounts of slab cracking for a 4-year-old pavement. However, there are no identifiable trends in the cracking of the pavement sections.

- While the serviceability data indicate no significant differences in the rideability of the pavement sections, the roughness data indicate that WI 4-4 (containing both longitudinal and transverse drains, but no joint sealant) is the smoothest riding section. WI 4-1 is the roughest pavement section; it is sealed but contains no positive drainage features.

Wisconsin 5 (STH 50, Kenosha)

An experimental project constructed in 1988 on STH 50 near Kenosha consists of six 10-in (254-mm) JPCP sections with AC shoulders constructed on a 6-in (152-mm) dense-graded aggregate base. Transverse joints are spaced at 12-13-19-18-ft (3.7-4.0-5.8-5.5-m) intervals and do not contain dowel bars. The design features of interest on this project include joint sealing and different types of drainage. All sections contain a 14-ft (4.3-m) wide outside traffic lane. The experimental design matrix for this project is shown in table 62.

Traffic Loadings

This pavement is a four-lane divided roadway that currently carries a two-way ADT of 9,600 vehicles per day, including 14 percent heavy trucks. Since being opened to traffic in 1988, it has sustained an estimated 1.4 million 18-kip (80-kN) ESAL applications in the outer traffic lane.

Performance

The performance data for the WI 5 sections are summarized in table 63. The same information is illustrated in figure 62.

Table 62. Experimental design matrix for WI 5.

		Type of Drainage		
		Longitudinal Fin Drain[1]	Longitudinal Pipe Drain[2]	No Edge Drain
10-in (254-mm) JPCP 12-13-19-18-ft (3.7-4.0-5.8-5.5-m) Skewed Joints No Dowels 6-in (152-mm) AGG	Silicone Sealant	WI 5-1	WI 5-3	WI 5-5
	No Sealant	WI 5-2	WI 5-4	WI 5-6

[1] 12-in by 1-in (305-mm by 25-mm) fin-type discharging to slope outfalls.
[2] 4-in (102-mm) fabric-wrapped, perforated PVC pipe discharging to slope outfalls.
All sections have 14-ft (4.3-m) outer lane and AC shoulders.

Overall, these pavement sections are in good condition, being only 4 years old at the time of survey. However, a few interesting observations can be made regarding their relative performance. For example, sections with longitudinal pipe drains have the most faulting of any of the pavement sections. In fact, the faulting on these sections is considered quite high for such a new pavement. Also, the sections with longitudinal pipe drains also show the most spalling, and one section is the only pavement to exhibit any transverse slab cracking. At this time, there do not appear to be any differences between the performance of the sections with fin drains and the sections without any drainage features.

While some spalling has developed on these sections, it is not a significant amount. Surprising, the sections with silicone joint sealant are showing slightly more spalling than the nonsealed sections.

Only one section (WI 5-5) is exhibiting any longitudinal cracking. This cracking occurs at the outer edge of the outer traffic lane and meanders toward the centerline between the two traffic lanes.

Roughness and Serviceability

The roughness and serviceability data for the WI 5 sections are illustrated in figure 63. The roughness values for these sections are clustered closely together, as are the serviceability ratings. No apparent trends present themselves regarding the rideability of the various pavement sections.

Summary

Overall, the WI 5 sections are in good condition after 4 years of service. None of the sections are exhibiting any major structural deterioration, and each section is providing good rideability. However, the faulting level is excessive on some sections after only 4 years of service. At this time, the sections with pipe drains are showing higher levels of faulting than the both the nondrained sections and the sections with

Table 63. Summary of 1992 outer lane performance data for WI 5.

			Type of Drainage: Long. Fin Drain	Long. Pipe Drain	No Edge Drain	Average
Age at Survey, years			4	4	4	4
ESAL's, millions			1.4	1.4	1.4	1.4
10-in (254-mm) JPCP 12-13-19-18-ft (3.7-4.0-5.8-5.5-m) Skewed Joints No Dowels 6-in (152-mm) AGG	**Silicone Sealant**		WI 5-1	WI 5-3	WI 5-5	
		Faulting, in	0.03	0.06	0.04	0.04
		Det. Tr. Crks/mi	0	0	0	0
		% Slabs Cracked	0	0	0	0
		Long. Cracks, ft/mi	0	0	545	182
		% Joints Spalled	8	12	6	9
		PSR	3.8	3.8	3.8	3.8
		IRI, in/mi	138	147	133	139
	No Sealant		WI 5-2	WI 5-4	WI 5-6	
		Faulting, in	0.05	0.06	0.03	0.05
		Det. Tr. Crks/mi	0	11	0	4
		% Slabs Cracked	0	3	0	1
		Long. Cracks, ft/mi	0	0	0	0
		% Joints Spalled	3	9	8	7
		PSR	3.7	3.9	3.7	3.8
		IRI, in/mi	132	142	142	139
Average		Faulting, in	0.04	0.06	0.04	
		Det. Tr. Crks/mi	0	5	0	
		% Slabs Cracked	0	2	0	
		Long. Cracks, ft/mi	0	0	272	
		% Joints Spalled	6	10	7	
		PSR	3.8	3.8	3.8	
		IRI, in/mi	135	144	138	

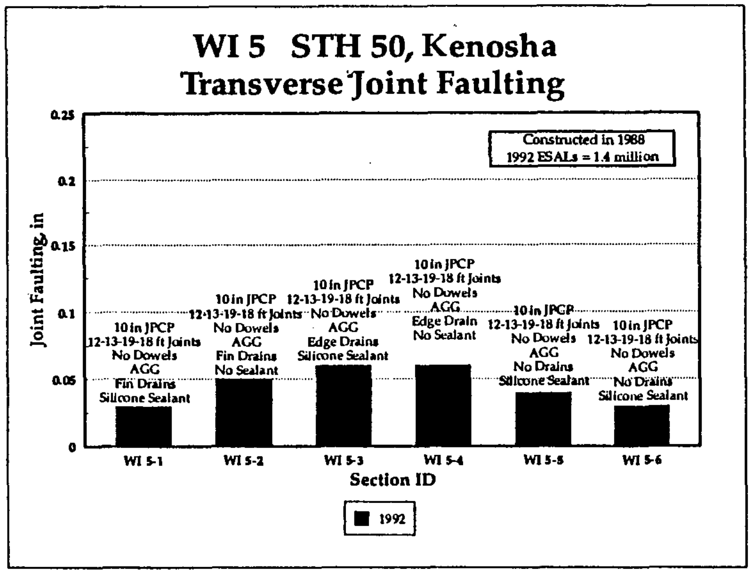
All sections have 14-ft outer lane and AC shoulders.

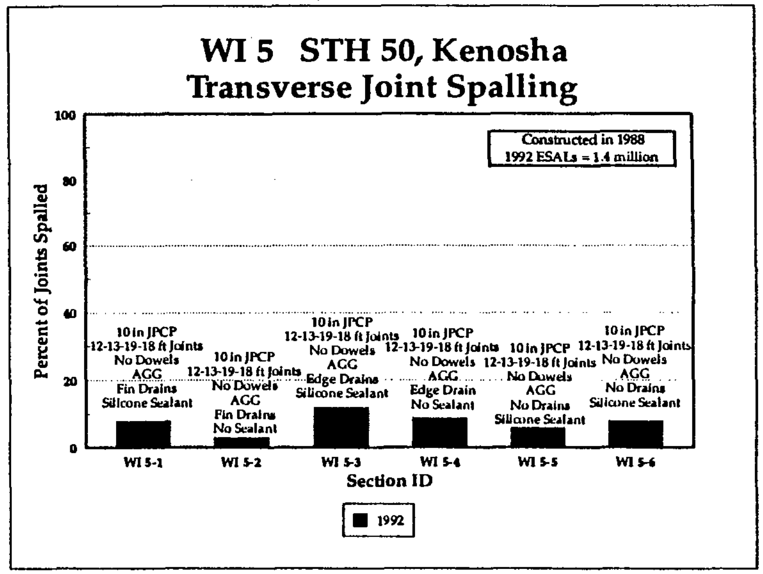
1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km

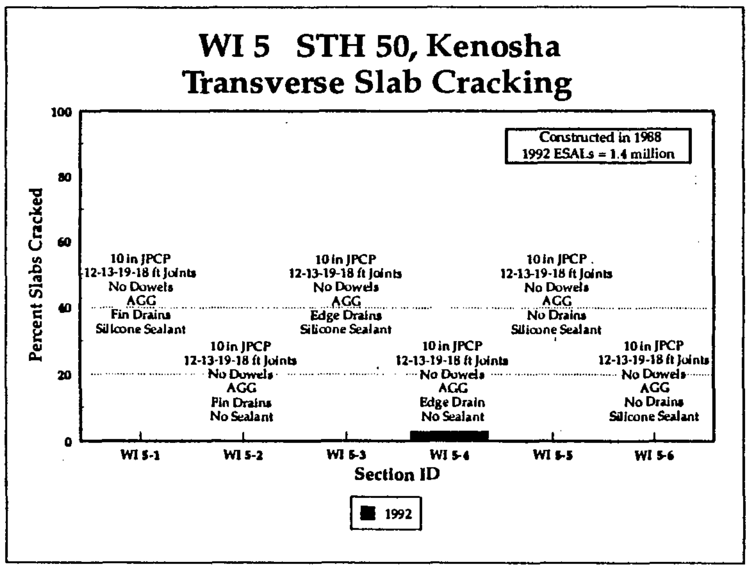
fin drains. In addition, the sections with silicone joint sealants are showing slightly more spalling than the nonsealed sections, although the differences are not considered significant.

Wisconsin 6 (STH 29, Green Bay)

This experimental project is located on STH 29 west of Green Bay. Built in 1988, the project consists of four experimental sections. All four sections are 10-in (254-mm) JPCP designs with 12-13-19-18-ft (3.7-4.0-5.8-5.5-m) joint spacing. Each section is constructed on a 4-in (102-mm) permeable aggregate base. Longitudinal collector drains consist of a 6-in (152-mm) perforated PVC pipe placed in a 12-in by 12-in (305-mm by 305-mm) fabric-lined trench filled with open-graded crushed aggregate.







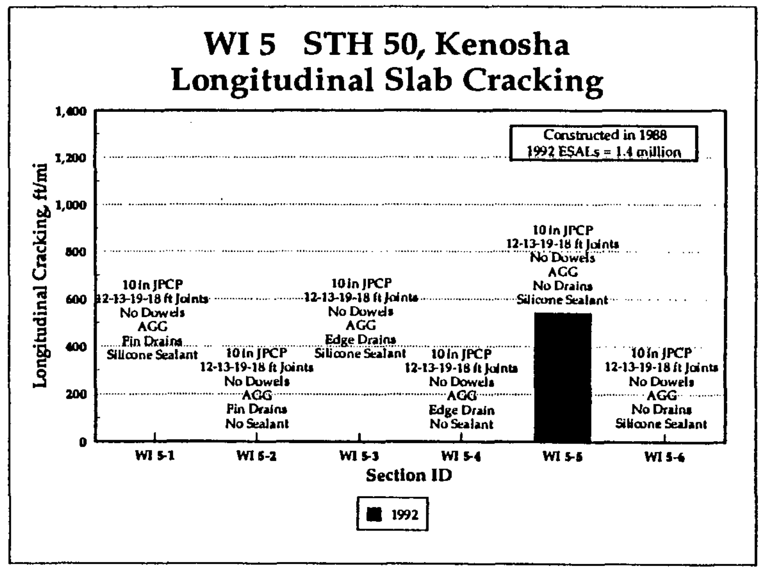
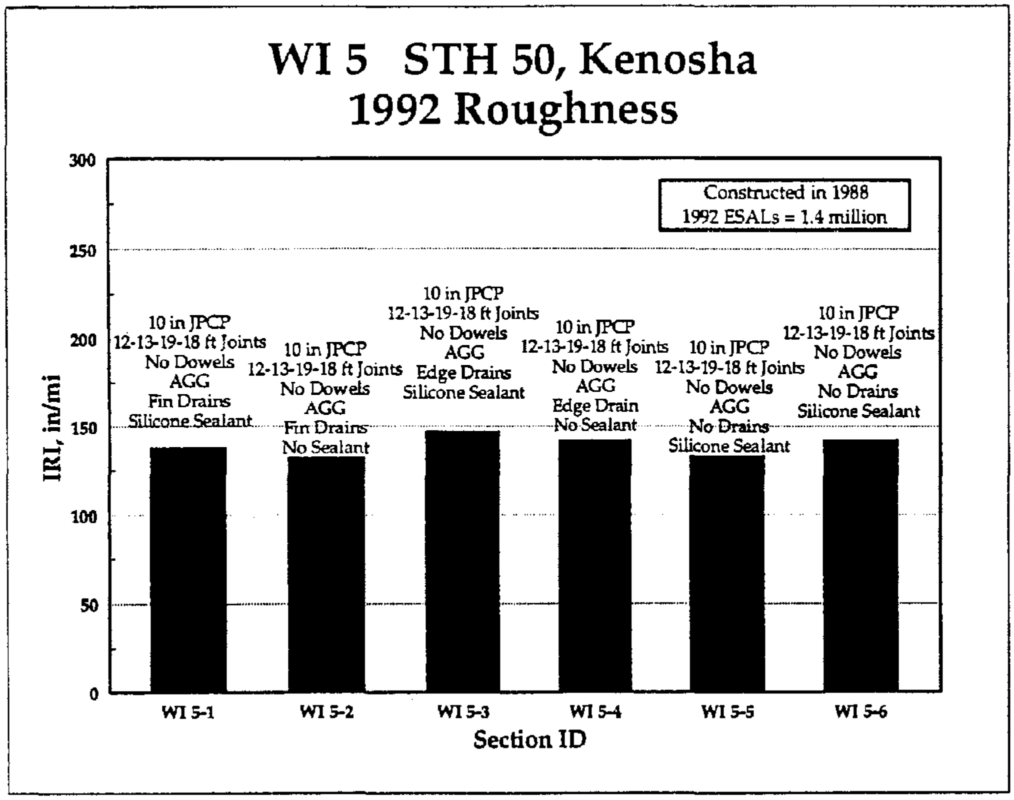


Figure 62. Performance summary for WI 5.



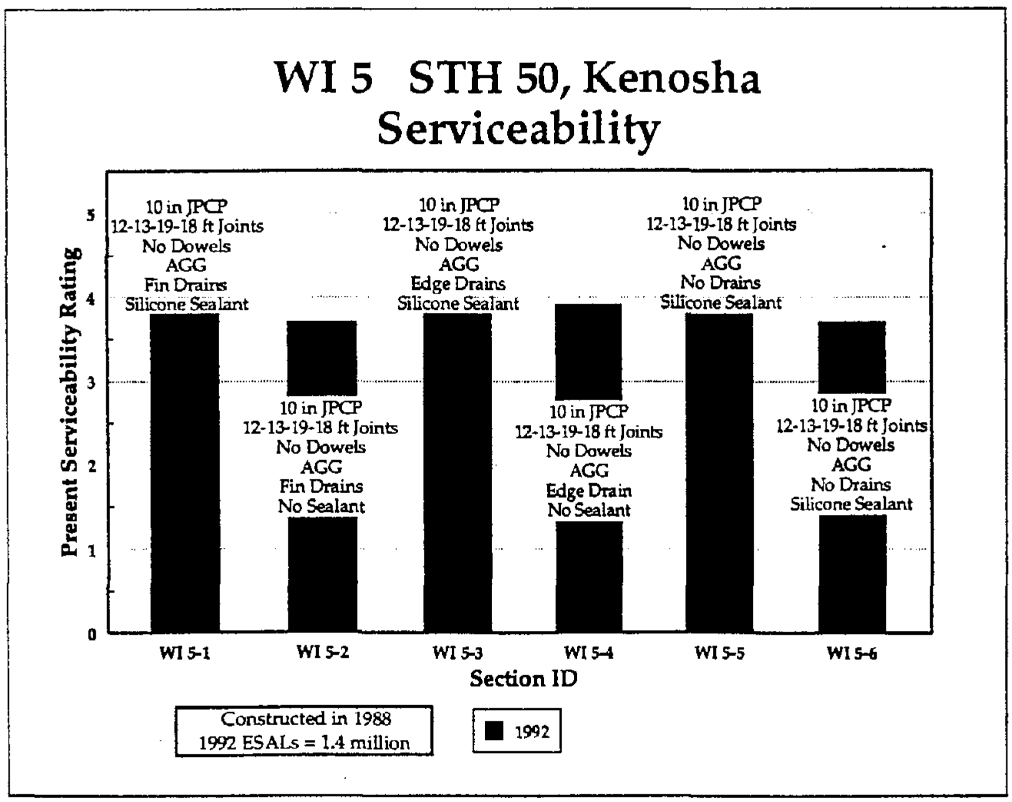


Figure 63. Roughness and serviceability for WI 5.

The experimental design factors that are being evaluated under this study are dowel bars and joint sealant. Where used, the joint sealant material is a preformed compression seal and the dowel bars are 1.5-in (38-mm) in diameter and coated with epoxy. Table 64 shows the design features evaluated in this project.

Table 64. Experimental design matrix for WI 6.

	10-in (254-mm) JPCP 12-13-19-18-ft (3.7-4.0-5.8-5.5-m) Skewed Joints 4-in (102-mm) PAGG Longitudinal Edge Drains	
	1.5-in (38-mm) Dowels	**No Dowels**
Preformed Joint Seal	WI 6-3	WI 6-2
No Joint Sealant	WI 6-4	WI 6-1

All sections contain a 4-in (102-mm) aggregate subbase.
All sections have 14-ft (4.3-m) outer lane.
WI 6-1 and 6-3 have tied PCC shoulders; other sections have AC shoulders.

Traffic Loadings

This four-lane divided roadway currently carries a two-way ADT of 14,200 vehicles per day, including 30 percent heavy trucks. Since its construction in 1988, this pavement has carried approximately 4.2 million 18-kip (80-kN) ESAL applications in the outer traffic lane.

Performance

The performance data for these sections are given in table 65 and in figure 64. This information clearly shows the benefits of dowel bars in reducing joint faulting, even after only 4 years of service. The doweled sections are showing no faulting, whereas both of the nondoweled sections are exhibiting significant levels of faulting for a 4-year-old pavement.

The preformed compression seal is apparently working well, as the sections with sealed joints are displaying less joint spalling than the nonsealed joints. In particular, WI 6-1 is exhibiting a substantial amount of medium- and high-severity joint spalling. Although some sections are showing some faulting and spalling, no sections are displaying any transverse or longitudinal cracking.

Roughness and Serviceability

Figure 65 illustrates the roughness and serviceability data for these sections. The roughness data indicate that these sections are very smooth with little perceptible differences between pavement sections. The serviceability data also indicate the excellent rideability furnished by these sections.

Table 65. Summary of 1992 outer lane performance data for WI 6.

		10-in (254-mm) JPCP 12-13-19-18-ft (3.7-4.0-5.8-5.5-m) Skewed Jts 4-in (102-mm) PAGG Longitudinal Edge Drains		Average
		1.5-in (38-mm) Dowels	**No Dowels**	
Age at Survey, years		4	4	4
ESAL's, millions		4.2	4.2	4.2
		WI 6-3	WI 6-2	
Preformed Joint Seals	Faulting, in	0.00	0.04	0.02
	Det. Tr. Crks/mi	0	0	0
	% Slabs Cracked	0	0	0
	Long. Cracks, ft/mi	0	0	0
	% Joints Spalled	0	2	1
	PSR	3.8	3.9	3.8
	IRI, in/mi	81	77	90
		WI 6-4	WI 6-1	
No Joint Sealant	Faulting, in	0.00	0.07	0.04
	Det. Tr. Crks/mi	0	0	0
	% Slabs Cracked	0	0	0
	Long. Cracks, ft/mi	0	0	0
	% Joints Spalled	2	18	10
	PSR	3.8	4.0	3.9
	IRI, in/mi	102	88	84
Average	Faulting, in	0.00	0.06	
	Det. Tr. Crks/mi	0	0	
	% Slabs Cracked	0	0	
	Long. Cracks, ft/mi	0	0	
	% Joints Spalled	1	10	
	PSR	3.8	4.0	
	IRI, in/mi	91	82	

All sections contain a 4-in (102-mm) aggregate subbase.
All sections have 14-ft (4.3-m) outer lane.
WI 6-1 and 6-3 have tied PCC shoulders, others have AC shoulders.

1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km

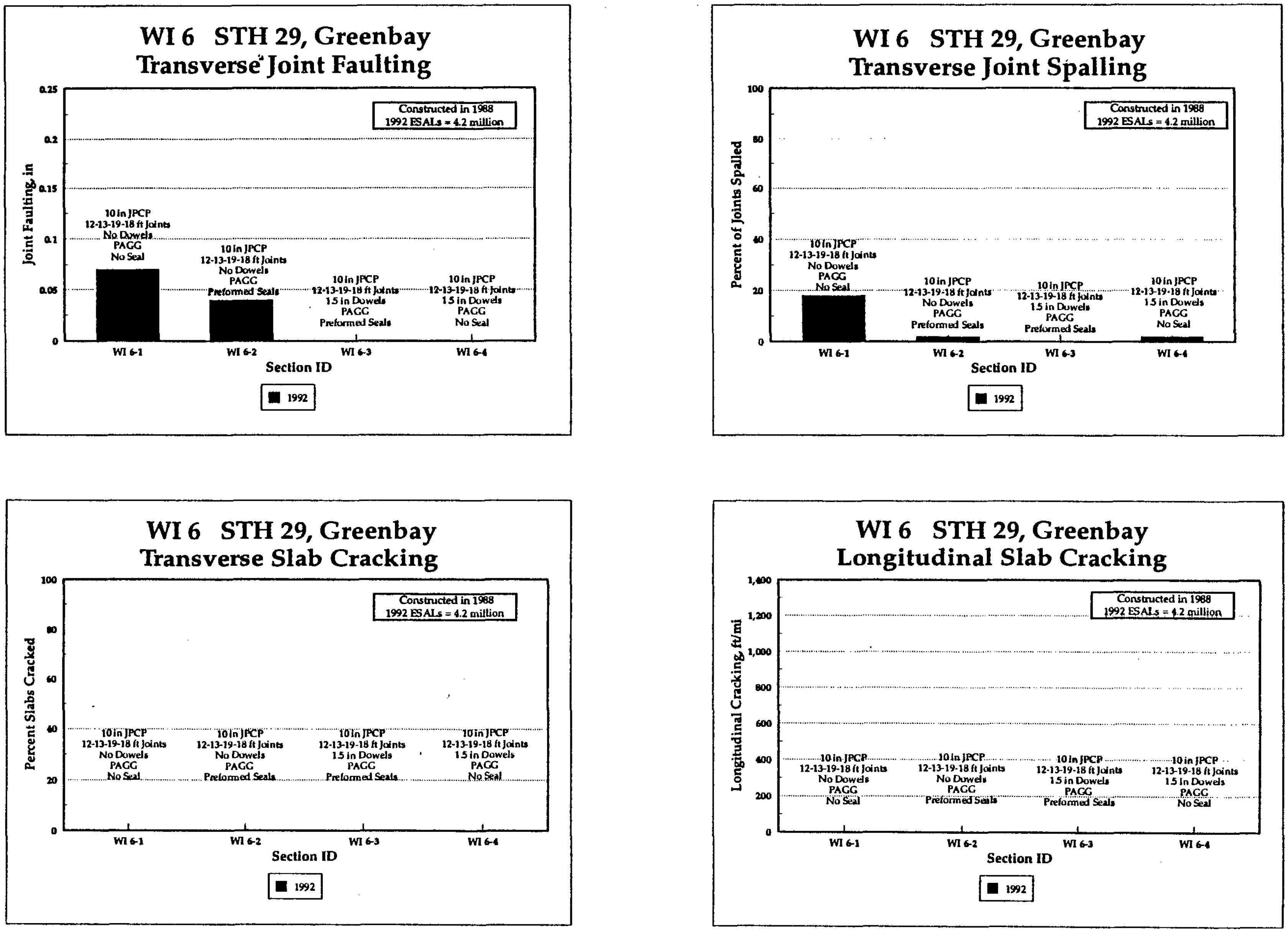
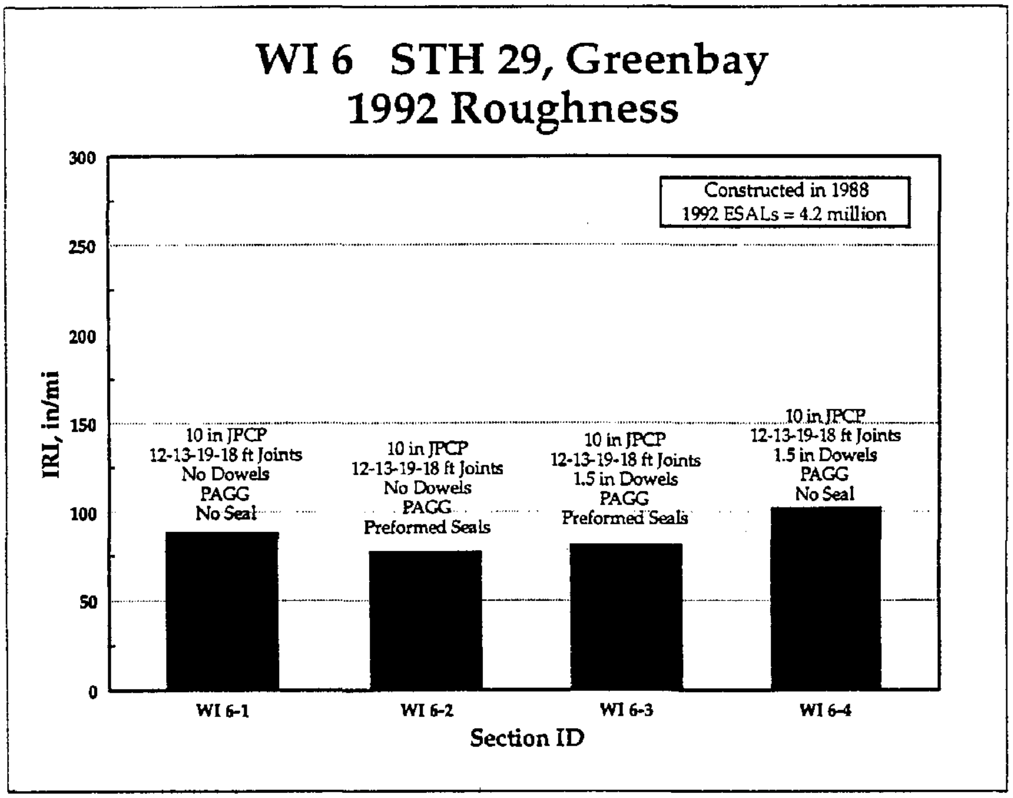


Figure 64. Performance summary for WI 6.



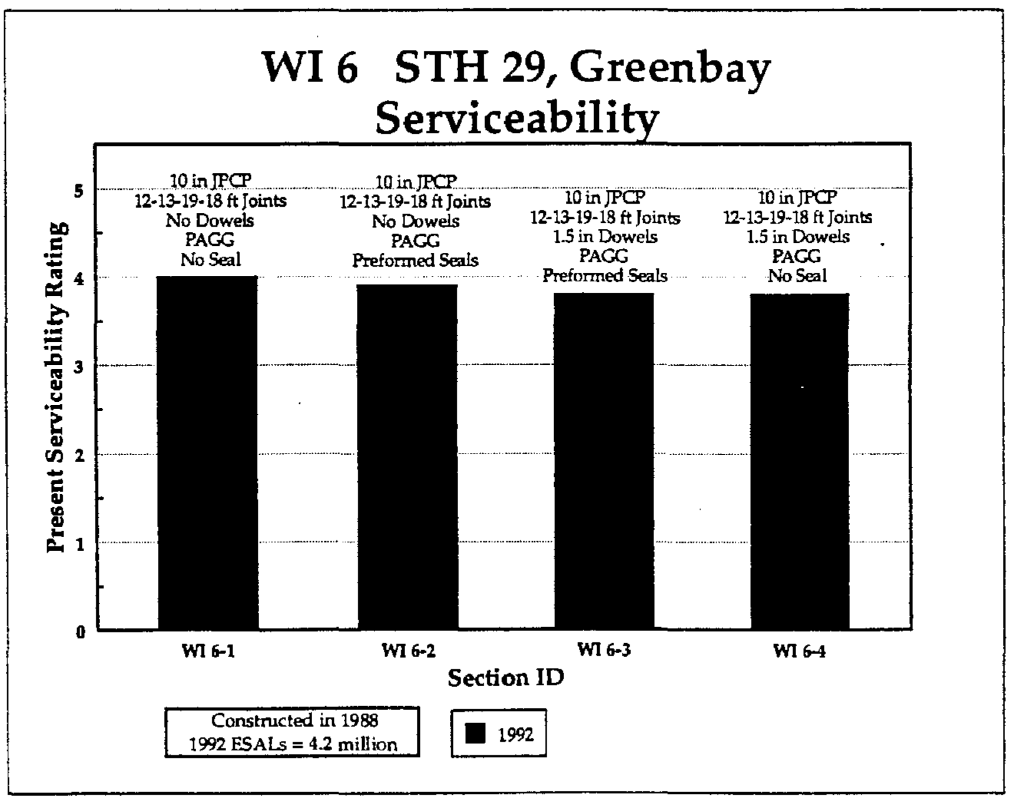


Figure 65. Roughness and serviceability for WI 6.

Summary

A summary of the performance of the WI 6 concrete pavement sections is given below:

- Overall, these pavement sections are in adequate condition, although some faulting does exist. No structural deterioration has occurred and all sections are quite smooth and providing good serviceability.
- The doweled sections are showing no faulting at all, whereas both nondoweled sections are exhibiting significant levels of faulting for a 4-year-old pavement.
- After only 4 years, the performance data show that the sections whose joints are sealed with preformed compression seals are displaying less spalling than the sections with nonsealed joints. In particular, WI 6-1 is exhibiting a substantial amount of medium- and high-severity joint spalling.

Performance Summary of Projects in Wet-Nonfreeze Zone

Four States in the wet-nonfreeze environmental region contribute projects to the study: California, Florida, Georgia, and North Carolina. The performance of the projects in these States is discussed in this section.

The projects in this environmental region have a Corps of Engineers Freezing Index of 0, a Thornthwaite Moisture Index ranging from 20 to 40, and annual precipitation ranging from 44 in (1118 mm) to 59 in (1499 mm). The highest average monthly maximum temperature is about 90 °F (32 °C), whereas the lowest average monthly minimum temperature ranges from 29 °F (-2 °C) to 50 °F (10 °C).

<u>California 3 (U.S. 101, Geyserville)</u>

In 1975, the California Department of Transportation constructed an experimental project on U.S. 101 near Geyserville to study the effects of shoulder type on pavement performance.[21] Seven different sections were constructed, including sections with tied PCC shoulders, nontied PCC shoulders, and various types of asphalt concrete shoulders. Some of these sections have sealed transverse joints, in contrast to the then-current practice in California of not sealing joints.

Common to these pavements is a 9-in (229-mm) JPCP slab over a 5.4-in (137-mm) CTB and a 6-in (152-mm) aggregate subbase. The transverse joints are spaced at 12-13-19-18-ft (3.7-4.0-5.8-5.5-m) intervals and do not contain dowel bars. The subgrade soil classification varies from an AASHTO A-4 to an AASHTO A-6 material. The tied PCC shoulder design consists of 22-in (560-mm) long, No. 4 (13-mm) bars at 30-in (760-mm) centers. The design matrix for this project is shown in table 66.

Table 66. Experimental design matrix for CA 3.

		9-in (229-mm) Nondoweled JPCP 5.4-in (137-mm) CTB (5% cement) 12-13-19-18-ft (3.7-4.0-5.8-5.5-m) Joints		
		Tied PCC Shoulder	Nontied PCC Shoulder	AC Shoulder
Transverse Joint Seal Type	Preformed	CA 3-1* CA 3-6	CA 3-3 CA 3-8	
	None	CA 3-2* CA 3-7	CA 3-5* CA 3-10	CA 3-4 CA 3-9

* 1987 section
All sections have 6-in (152-mm) aggregate subbase and A-4 subgrade.

Traffic Loadings

This four-lane roadway currently carries a two-way ADT of 17,000 vehicles per day, including 9 percent trucks. Through 1987, the outer lane of the highway had sustained 3.6 million ESAL applications. The section had experienced 5.7 million ESAL applications through 1992.

Performance

The performance of these sections is summarized in table 67 and in figure 66. The effect of joint sealing can be examined by comparing the performance of CA 3-1 and CA 3-6 with CA 3-2 and CA 3-7, and by comparing CA 3-3 and CA 3-8 with CA 3-5 and CA 3-10. For the first group of sections with a tied PCC shoulder, the sections that are sealed are performing better than those that are not sealed. They have lower levels of faulting and joint spalling, although the amount of joint spalling that has developed on the nonsealed sections is not significant. Transverse cracking is about the same for both groups of sections.

For the sections constructed with nontied PCC shoulders, the sealed sections show no spalling while the nonsealed exhibit a small quantity of spalled joints. The sealed sections also show less transverse cracking, although the faulting is about the same for the sealed and nonsealed sections.

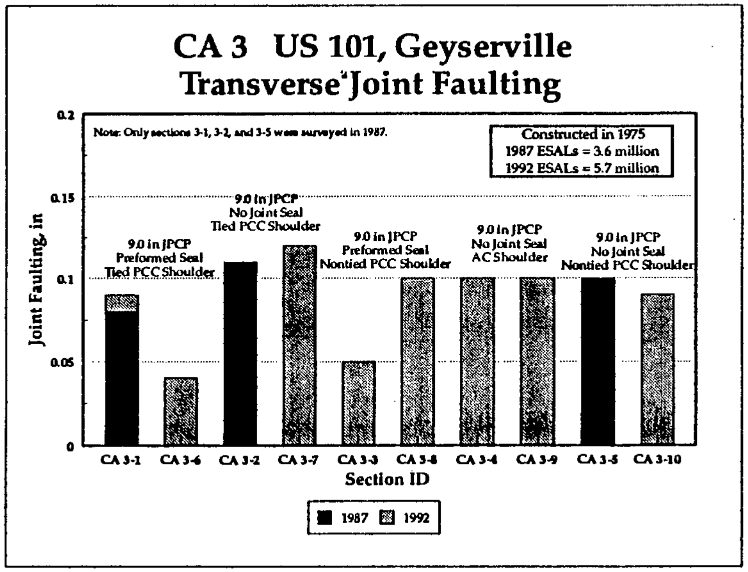
Evaluating the effect of shoulder type revealed that sections with tied PCC shoulders have fewer transverse cracks than other shoulder types. The additional support provided to the slab by the tied shoulder apparently was effective in reducing the magnitude of the critical edge stress, thereby reducing the amount of transverse cracking. In addition, the tied shoulder appeared to have a slight effect in reducing joint faulting. The sections with AC shoulders exhibited fewer transverse cracks than the sections without tied PCC shoulders and about the same number of transverse cracks as the sections with tied PCC shoulders. This phenomenon is likely

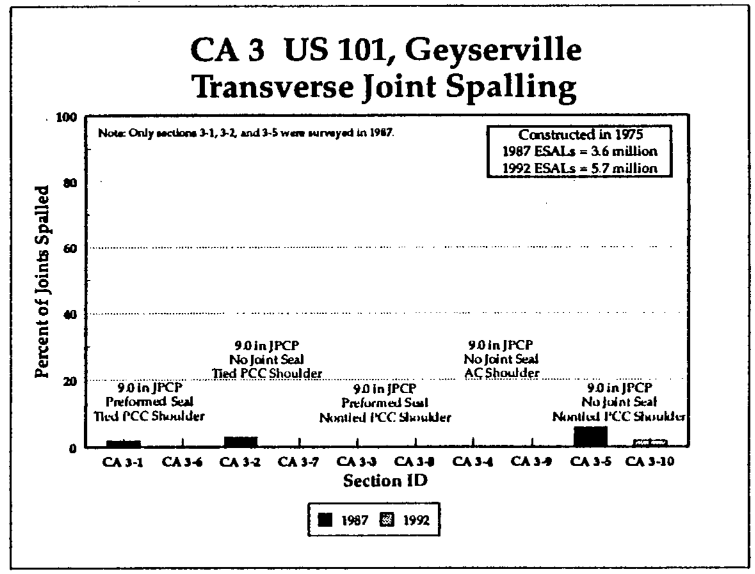
Table 67. Summary of 1987 and 1992 outer lane performance data for CA 3.

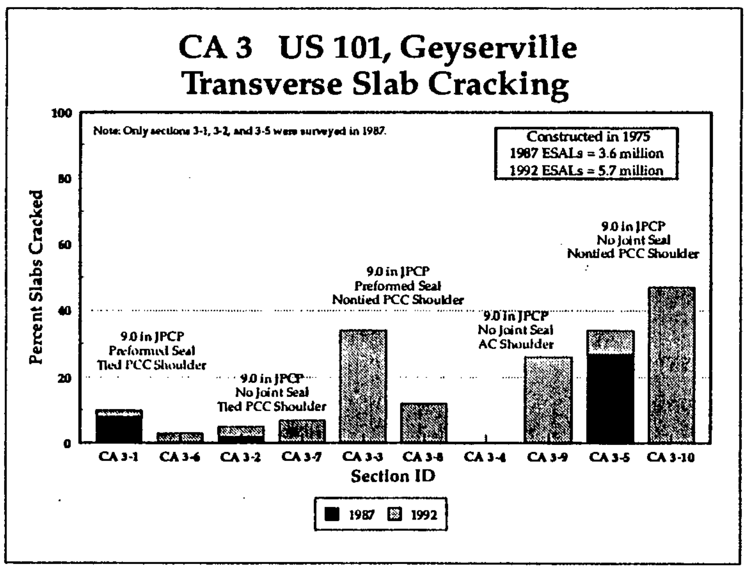
			9-in (229-mm) Nondoweled JPCP 5.4-in (137-mm) CTB (5% cement) 12-13-19-18-ft (3.7-4.0-5.8-5.5-m) Joints							
			Tied PCC Shoulder			Nontied PCC Shoulder			AC Shoulder	
			1987 Survey	1992 Survey		1987 Survey	1992 Survey		1992 Survey	
Age at Survey, years			12	17		12	17		17	
ESAL's, millions			3.6	5.7		3.6	5.7		5.7	
Transverse Joint Seal Type	Pre-formed		CA 3-1	CA 3-1	CA 3-6		CA 3-3	CA 3-8		
		Faulting, in	0.08	0.09	0.04		0.05	0.10		
		Det. Tr. Crks/mi	27	33	9		135	40		
		% Slabs Cracked	8	10	3		34	12		
		Long. Cracks, ft/mi	0	0	0		25	0		
		% Joints Spalled	2	0	0		0	0		
		PSR	3.6	3.4	3.7		3.7	3.4		
		IRI, in/mi	—	167	111		126	121		
	None		CA 3-2	CA 3-2	CA 3-7	CA 3-5	CA 3-5	CA 3-10	CA 3-4	CA 3-9
		Faulting, in	0.11	0.09	0.12	0.10	0.08	0.09	0.10	0.10
		Det. Tr. Crks/mi	6	18	24	123	135	195	0	88
		% Slabs Cracked	2	5	7	27	34	47	0	26
		Long. Cracks, ft/mi	0	0	0	34	34	0	0	0
		% Joints Spalled	3	3	2	6	6	2	0	0
		PSR	3.9	3.5	3.5	3.8	3.5	3.5	3.8	3.0
		IRI, in/mi	—	129	147	—	134	116	150	144

All sections have 6-in aggregate subbase and A-4 subgrade.

1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km







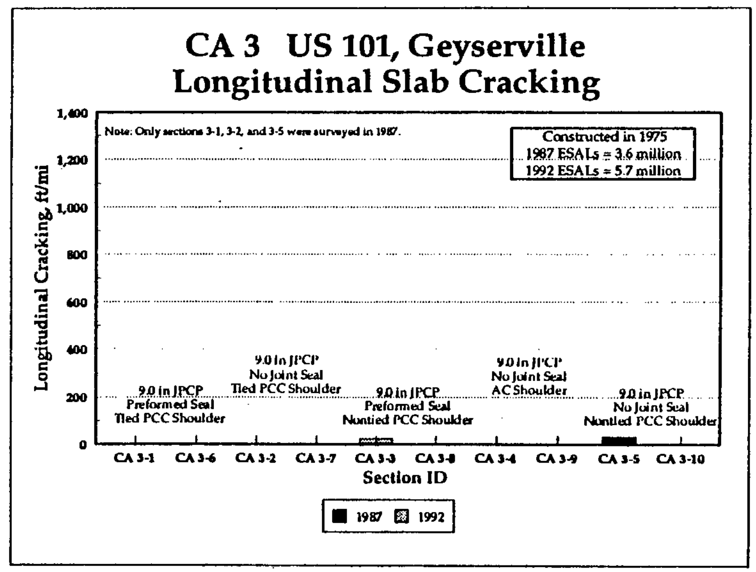


Figure 66. Performance summary for CA 3.

related to the ineffective shoulder tie bar design system. However, the sections with AC shoulders had more faulting than the sections with tied PCC shoulders.

Roughness and Serviceability

Figure 67 shows the roughness and serviceability data for the CA 3 sections. The roughness data show that the smoothest pavement is section 3-6, which contains sealed joints and a tied PCC shoulder. However, several of the sections with nontied PCC shoulders are also providing a smooth-riding surface. Overall, the roughest group of pavement sections are those constructed with AC shoulders. Similar observations are apparent from an examination of the serviceability data.

The serviceability of the three sections surveyed in 1987 (CA 3-1, 3-2, and 3-5) has decreased since that survey. The biggest decrease occurred in section 3-2, which contains a tied PCC shoulder and no joint sealant.

5-Year Performance Trends

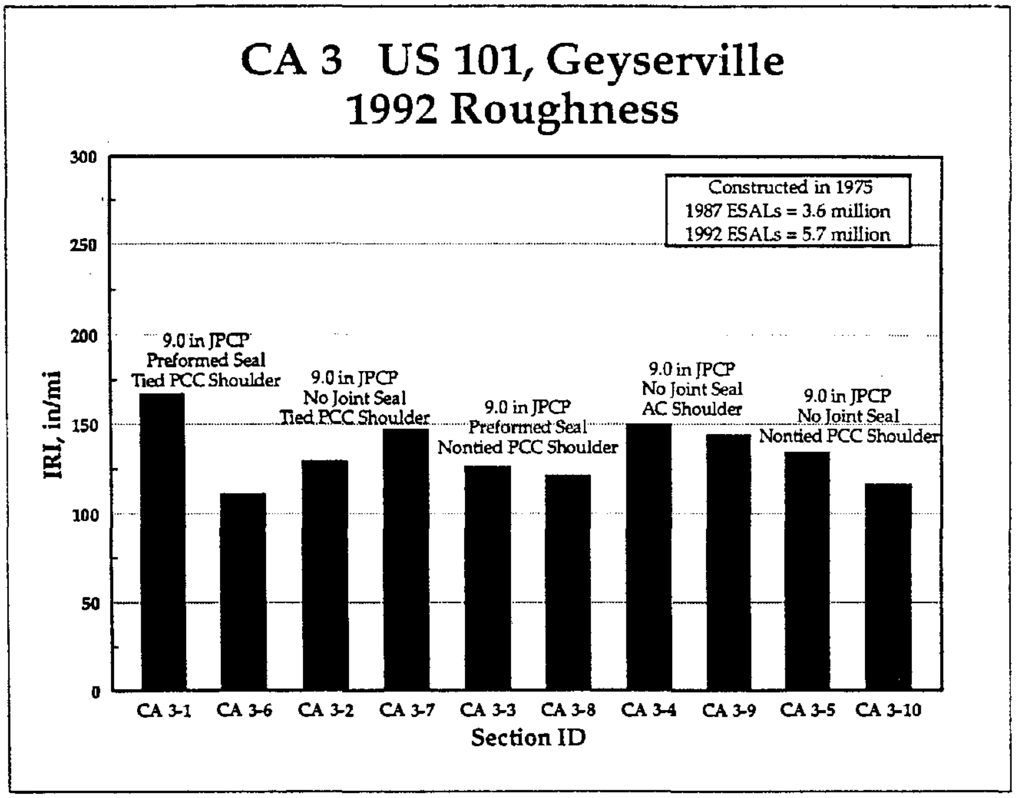
The 5-year performance trends for the three sections surveyed in 1987 (3-1, 3-2, and 3-5) are shown in figure 68. This figure shows that no major changes in distress have occurred over the 5-year period, even though figure 67 showed a decrease in overall rideability. Joint spalling and longitudinal cracking remain unchanged from the 1987 survey, and joint faulting shows only a minor variation. Transverse cracking has increased slightly for all three sections with the biggest increase in the section with a nontied PCC shoulder.

Summary

A summary of the performance of the CA 3 sections is provided below:

- The pavement sections with sealed transverse joints (preformed sealant) show slightly less faulting and spalling than the sections with nonsealed transverse joints. However, the spalling that occurred on these sections was not considered significant. The performance of the sealed sections was particularly good when constructed with a tied PCC shoulder.

- The performance of the sections with tied PCC shoulders was superior to that of the sections with nontied PCC shoulders. The sections with nontied shoulders showed significantly more slab cracking and slightly more transverse faulting than the sections with tied shoulders, demonstrating the benefits of providing lateral support to the slab. This agrees with the findings of a California Department of Transportation study on this project that indicated the best performance was provided by the tied and sealed section, in terms of faulting and cracking.[37] Curiously, however, the sections with AC shoulders performed better than the nontied PCC shoulders and almost as well as the sections with the tied PCC shoulders, which indicates that the shoulder tie bar design may be inadequate.



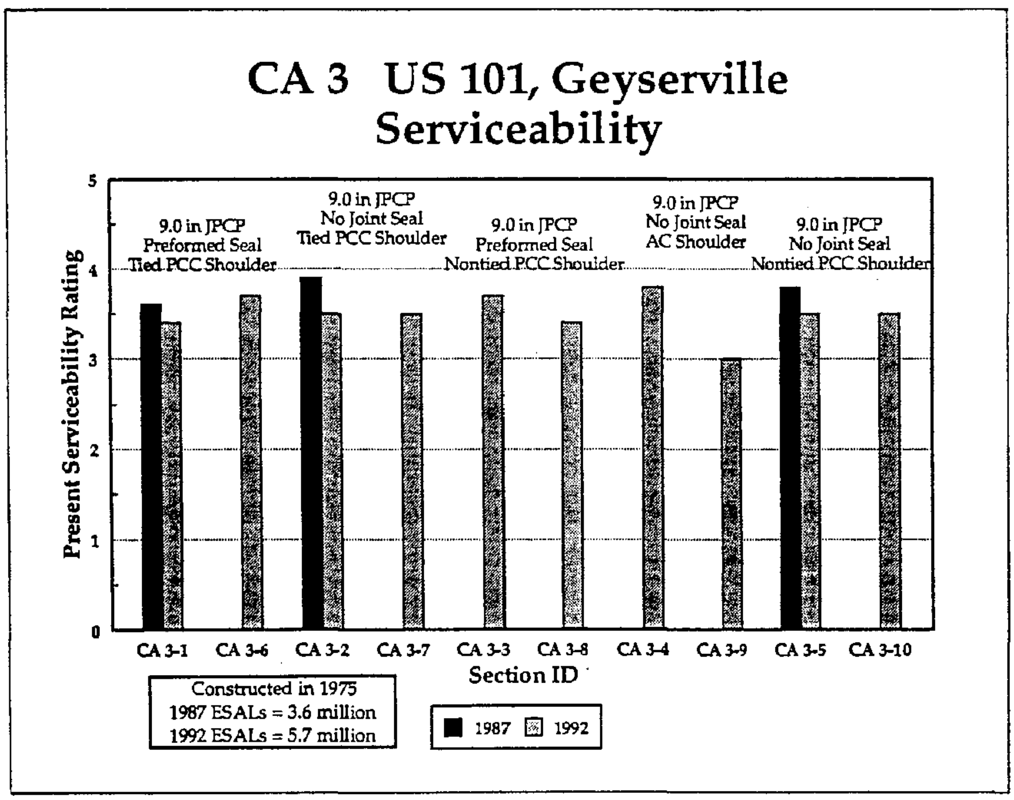


Figure 67. Roughness and serviceability for CA 3.

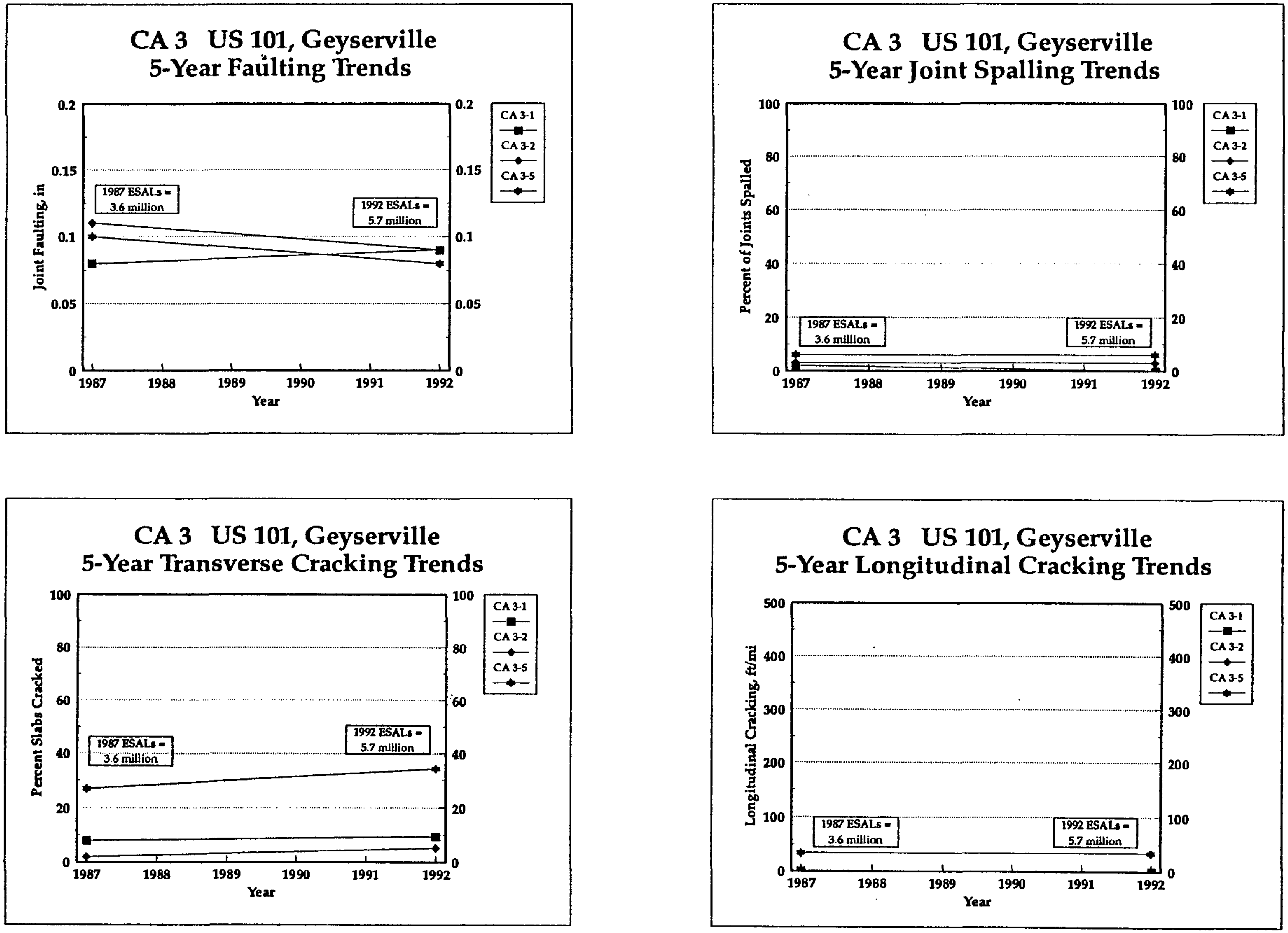


Figure 68. 5-year performance trends for CA 3.

- Overall, the roughest pavement sections were those constructed with AC shoulders. The smoothest pavement section was CA 3-6, which contained sealed transverse joints and a tied PCC shoulder. However, several sections with nontied PCC shoulders are also providing a smooth-riding surface.

- A decrease in serviceability was observed for the three sections that were surveyed in 1987 (CA 3-1, 3-2, and 3-5). The biggest decrease occurred in section 3-2, which contained a tied PCC shoulder and no joint sealant.

- An examination of the 5-year performance trends indicate virtually no changes in pavement distress since the 1987 survey. The biggest change was observed in section CA 3-5, which showed an increase in transverse cracking from 27 to 34 percent of the slabs.

<u>California 9 (I-680, Milpitas, CA)</u>

This experiment, located on I-680 in Milpitas, was conducted by the California Department of Transportation to evaluate the effect of joint sealing on the performance of concrete pavements.[21] Interstate 680 consists of four 12-ft (3.7-m) lanes in each direction with AC shoulders. The project was constructed in 1974 and included ten 1000-ft (305-m) sections, each consisting of 65 joints. Four types of joint seal materials were evaluated in this study:

- Sealflex 1401, a hot-poured material meeting Federal Specification SS-S-1401.
- Sealflex 39, a polyurethane sealant.
- Superseal 444, a hot-poured, polyvinylchloride (PVC) coal tar sealant.
- Preformed neoprene compression seal.

The experiment also includes a section containing a longitudinal edge drain and a control section containing no edge drain and no joint sealant. Table 68 shows the design matrix for the sections selected for evaluation. A review of these sections indicates that several of the materials or construction practices are no longer in common use. However, these sections were evaluated to determine the performance of the sections relative to one another.

Traffic Loadings

Since being opened to traffic in 1974, this eight-lane roadway has sustained 10.5 million ESAL applications in the outer traffic lane. It currently is subjected to a two-way ADT of 140,000 vehicles per day, including 4 percent heavy trucks.

Performance

Table 69 and figure 69 present the performance data for the outer traffic lane of the CA 9 sections. The six sections for which faulting measurements are available are all exhibiting significant levels of faulting (0.10 to 0.15 in (2.5 to 3.8 mm), and high-severity pumping was noted on each section. The section exhibiting the least amount

Table 68. Experimental design matrix for selected sections on CA 9.

		9-in (229-mm) nondoweled JPCP 5.4-in (137-mm) CTB (5% cement) 12-13-19-18-ft (3.7-4.0-5.8-5.5-m) Joints	
		No Drains	Drains
Joint Sealant Type	Polyurethane	9-2	
	Hot-Pour	9-1 (no reservoir) 9-3	
	PVC Coal Tar	9-4	
	Preformed	9-5 9-6	
	None	9-10	9-8

All sections contain fiber cord backer rod except 9-5 (none), 9-6 (none), and 9-1 (fiber welt backer rod).
All sections contain a 6-in (152-mm) aggregate subbase.

of faulting is the control section (9-10), which contains no joint seal and no drainage. The section with drains is exhibiting the greatest amount of faulting, although this can be attributed to the fact that the outlets were damaged by landscaping crews about 3 years after construction and were never repaired.[21]

The relative performance of the various joint seal types is difficult to determine. The sections sealed with preformed compression seal and PVC coal tar are showing the lowest amount of faulting of the sealed sections. However, after 18 years of service, very little spalling has developed on any of the sections, which prevents the identification of a sealant that is performing better than the others.

Although the control section (no seal, no drains) exhibited the lowest amount of faulting, it also shows the greatest amount of slab cracking (67 percent of the slabs). This level is far greater than any of the sealed sections or the drained section. The sections with hot-poured sealant show no slab cracking, followed closely by the section sealed with PVC coal tar. The sections sealed with preformed compression seals show the most slab cracking (9 and 24 percent, respectively) of the sealed sections. These data indicate that sealing joints reduces slab cracking, mainly from the reduction of erosion and loss of support at the joints.

An examination of the joint sealant condition data indicates that all of the sealants, with the exception of the preformed compression seal, are showing failures at nearly every joint. These failures include such items as adhesion loss, cohesion failure, and hardening. The sections with compression seals, on the other hand, show very few distresses or failures.

Table 69. Summary of 1992 outer lane performance data for CA 9.

			9-in (229-mm) nondoweled JPCP 5.4-in (137-mm) CTB (5% cement) 12-13-19-18-ft (3.7-4.0-5.8-5.5-m) Joints		
			No Drains		Drains
			1992 Survey		1992 Survey
Age at Survey, years			18		18
ESAL's, millions			10.5		10.5
Joint Sealant Type	Poly-urethane		CA 9-2		
		Faulting, in	0.15		
		Det. Tr. Crks/mi	37		
		% Slabs Cracked	11		
		Long. Cracks, ft/mi	0		
		% Joints Spalled	0		
		PSR	3.2		
		IRI, in/mi	148		
	Hot-Pour		CA 9-3	CA 9-1	
		Faulting, in	0.14	—	
		Det. Tr. Crks/mi	0	0	
		% Slabs Cracked	0	0	
		Long. Cracks, ft/mi	0	0	
		% Joints Spalled	0	2	
		PSR	3.2	—	
		IRI, in/mi	154	—	
	PVC Coal Tar		CA 9-4	CA 9-7	
		Faulting, in	0.12	—	
		Det. Tr. Crks/mi	6	44	
		% Slabs Cracked	2	13	
		Long. Cracks, ft/mi	0	0	
		% Joints Spalled	0	0	
		PSR	3.5	—	
		IRI, in/mi	161	—	
	Preform		CA 9-5	CA 9-6	
		Faulting, in	0.12	—	
		Det. Tr. Crks/mi	36	83	
		% Slabs Cracked	9	24	
		Long. Cracks, ft/mi	0	0	
		% Joints Spalled	2	0	
		PSR	3.1	—	
		IRI, in/mi	163	—	
	None		CA 9-10		CA 9-8
		Faulting, in	0.10		0.17
		Det. Tr. Crks/mi	272		11
		% Slabs Cracked	67		3
		Long. Cracks, ft/mi	96		0
		% Joints Spalled	2		0
		PSR	2.9		3.3
		IRI, in/mi	153		212

All sections contain a 6-in aggregate subbase.

1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km

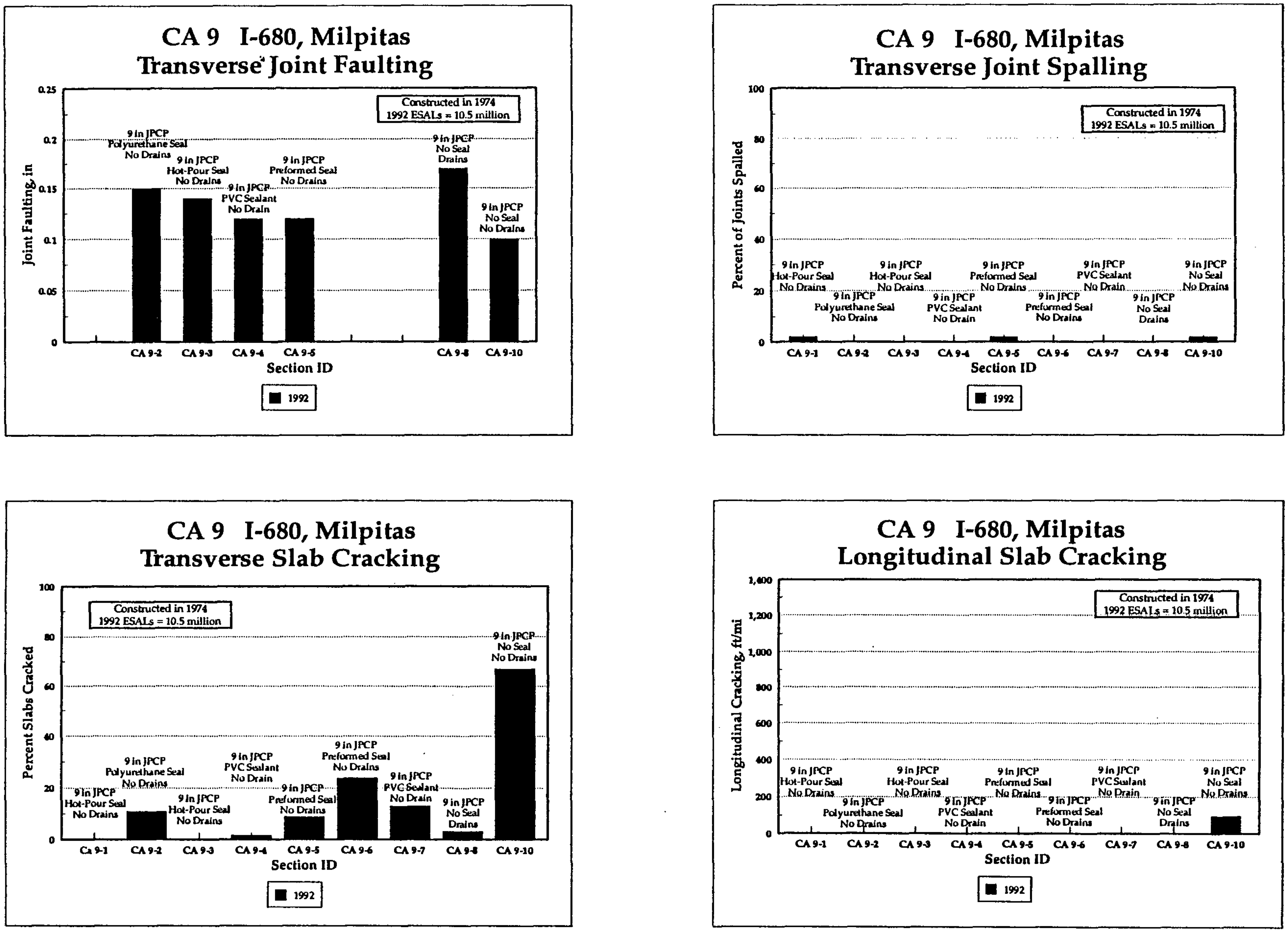


Figure 69. Performance summary for CA 9.

Overall, some of the sealed sections are performing better than the control section, particularly in terms of transverse cracking. However, even after 18 years, it remains to be seen what effects the sealants will have on joint spalling.

Roughness and Serviceability

Figure 70 shows the roughness and serviceability data for the CA 9 sections. The roughest section is the one constructed with drains, which also had the greatest amount of faulting. The smoothest riding section is CA 9-2, which contains a polyurethane sealant. However, with the exception of the drained section, the roughness data for all of the sections are not significantly different. These same trends are shown by the serviceability data.

Summary

The performance of the experimental sections constructed on I-680 is summarized below:

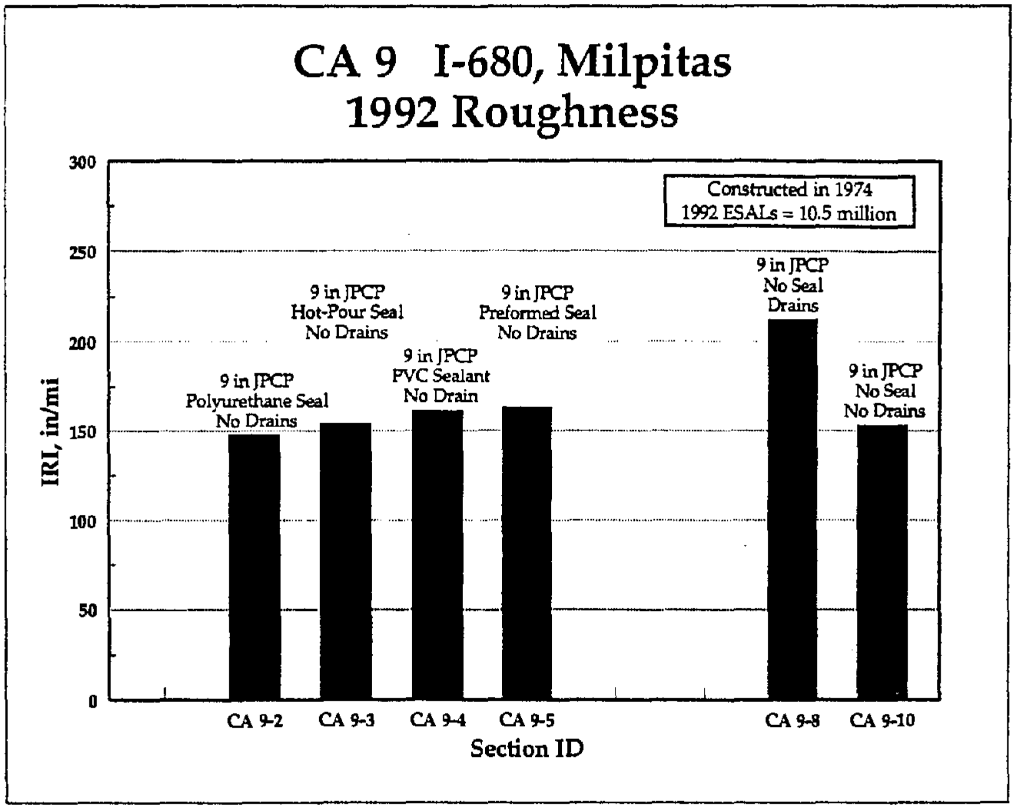
- Every section is exhibiting significant levels of faulting. The section showing the least amount of faulting is the control section (no seal, no drain), and the section showing the greatest amount of faulting is the drained section. However, the outlets for the drained section were damaged about 3 years after the pavement was constructed and never repaired. The sections sealed with the preformed compression seal and with the PVC coal tar are showing the lowest amount of faulting of the sealed sections.

- The relative performance of the various joint seal types is difficult to determine. Every section, regardless of sealant type, was displaying high-severity pumping in the form of stains on the shoulder. In addition, every sealant, with the exception of the preformed compression seal, was severely damaged at nearly every joint (the preformed seal showed only a few failures). However, very little spalling has developed on any of the sections after 18 years (including the nonsealed sections).

- While the control section (no seal, no drains) showed the lowest amount of faulting, it also displayed much more transverse slab cracking than any of the other sections. The sections with hot-poured sealant show no slab cracking, followed closely by the section sealed with PVC coal tar. The sections sealed with preformed compression seals show the most slab cracking of the sealed sections, probably a result of erosion of the CTB.

- Overall, considering all pavement distresses, some of the sealed sections are performing better than the control section.

- With the exception of the drained section, which was exhibiting very high roughness levels, all sections are exhibiting comparable levels of roughness.



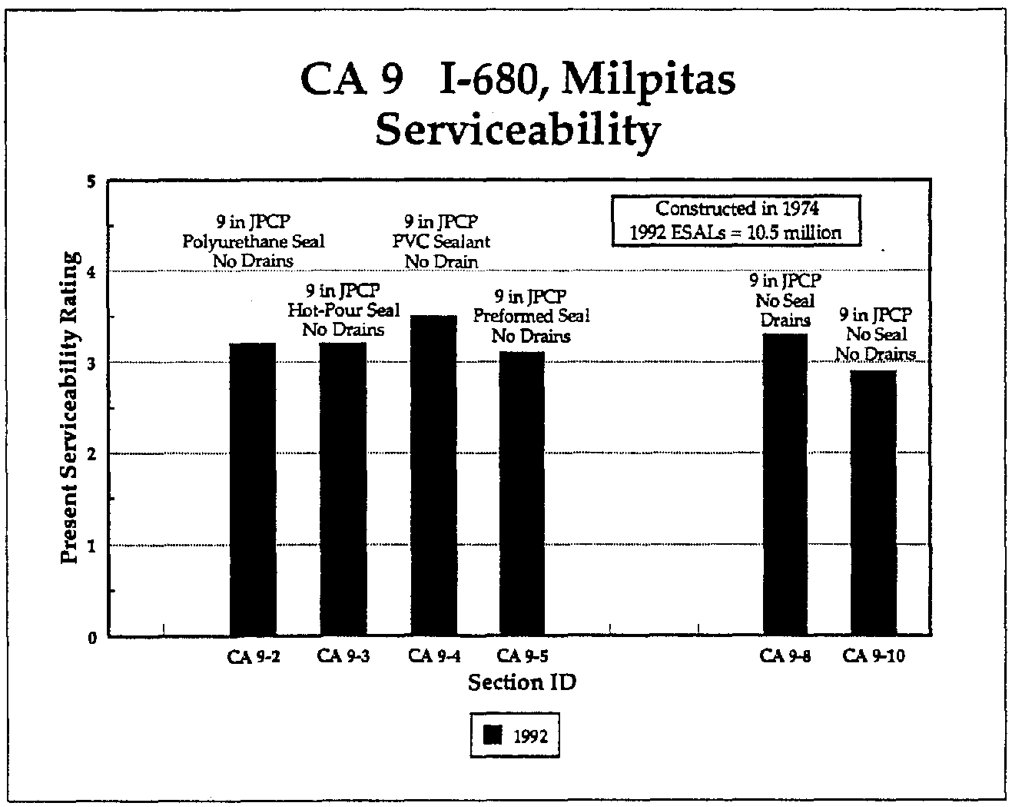


Figure 70. Roughness and serviceability for CA 9.

California 10 (U.S. 101, Ukiah)

This project is a single section that is located on U.S. 101 near Ukiah. It is a 9-in (229-mm) JPCP design constructed over a 4-in (102-mm) permeable asphalt-treated base. The transverse joints are spaced at 12-15-13-14-ft (3.7-4.6-4.0-4.3-m) intervals and do not contain dowel bars or joint sealant. The section was constructed in 1990 and is included in the study because of its permeable base design.

The design and performance data for this section are summarized in table 70. This pavement section is not exhibiting any structural deterioration, but it is exhibiting a rather high level of faulting (0.05 in [1.3 mm]) after only 2 years of service. Part of this faulting may be due to stripping of the asphalt from the aggregate in the PATB. An examination of the condition of the PATB by the California Department of Transportation noted that the stripping of the asphalt from the aggregate was severe at transverse joints.[23]

Table 70. Design and outer lane performance data for CA 10.

	CA 10
Pavement Type	JPCP
Year Built	1990
Thickness, in	9.0 (229 mm)
Joint Spacing, ft	12-15-13-14 (3.7-4.6-4.0-4.3 m)
Dowel Diameter, in	None
Base Type	4-in (102-mm) PATB (2.5% AC-20)
Subbase	None
Subgrade	A-6
Performance Data	1992 Survey
Age at Survey, years ESAL's, millions	2 1.0
Faulting, in Det. Tr. Crks/mi % Slabs Cracked Long. Cracks, ft/mi % Joints Spalled PSR IRI, in/mi	0.05 0 0 0 0 3.9 93

1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km

Florida 2 (I-75, Brandon)

Constructed in 1986, this single section is part of six-lane I-75 in Hillsborough County, near Brandon. The design of this section consists of a 13-in (330-mm) JPCP with a 14-ft (4.3-m) widened outside lane placed over a 6-in (152-mm) sand base course. The transverse joints are skewed, contain 1.25-in (32-mm) diameter, epoxy-coated dowel bars, and are spaced at 12-18-19-13-ft (4.9-5.5-5.8-4.0-m) intervals. The subgrade is classified as an AASHTO A-3 material.

Information on the design and performance of FL 2 is given in table 71. The pavement is still performing well after 6 years of service. No structural deterioration has occurred, and only a small amount of spalling is present. However, the level of faulting, while not critical, is considered somewhat high for a pavement of this age, although the overall rideability is still good. In comparing the 1987 and 1992 performance, it is observed that the faulting has increased and that some low-severity joint spalling has developed, but neither significantly detract from the performance of the pavement.

Table 71. Design and outer lane performance data for FL 2 and FL 3.

	FL 2		FL 3	
Pavement Type	JPCP		JPCP	
Year Built	1986		1982	
Thickness, in	13.0 (330-mm)		9.0 (229-mm)	
Joint Spacing, ft	12-18-19-13 (3.7-5.5-5.8-4.0-m)		16-17-23-22 (4.9-5.2-7.0-6.7-m)	
Dowel Diameter, in	1.25 (32-mm)		1.00 (25-mm)	
Base Type	6-in (152-mm) Sand		6-in (152-mm) LCB (10% cement)	
Subbase	None		None	
Subgrade	AASHTO A-3		AASHTO A-3	
Performance Data	1987 Survey	1992 Survey	1987 Survey	1992 Survey
Age at Survey, years	1	6	5	10
ESAL's, millions	2.5	9.5	5.9	13.0
Faulting, in	0.01	0.05	0.08	0.05
Det. Tr. Crks/mi	0	0	300	371
% Slabs Cracked	0	0	87	100
Long. Cracks, ft/mi	0	0	900	1037
% Joints Spalled	0	7	2	45
PSR	3.7	3.7	3.2	2.7
IRI, in/mi	—	93	—	190

FL 2 has a 14-ft outer lane.

1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km

Florida 3 (I-75, Bradenton)

This single section, constructed in 1982, is located on I-75 near Bradenton, in Manatee County. The design for this section is a 9-in (229-mm) JPCP with tied PCC shoulders, and random, skewed joints spaced at 16-17-23-22-ft (4.9-5.2-7.0-6.7-m) intervals. The transverse joints contain 1.00-in (25-mm) diameter, epoxy-coated dowels and are sealed with a silicone joint sealant. The base consists of 6 in (152 mm) of lean concrete, and the subgrade is an AASHTO A-3 material.

The design and performance data for this section is summarized in table 71. This section is exhibiting poor performance, in terms of excessive slab cracking and severe joint spalling, probably a result of the long joint spacing. All slabs within the survey section exhibit transverse cracking, and longitudinal cracking was found over 20 percent of the section. Many joints are badly spalled, and several had undergone joint repair since the 1987, which may explain why the joint faulting decreased from 1987. Because of the severe cracking and spalling, the rideability of the pavement is very poor. Since being surveyed in 1987, this pavement has continued to deteriorate, with an increase in transverse cracking and joint spalling, and a significant decrease in serviceability.

Florida 4 (U.S. 41, Ft. Meyers)

This experimental project, whose objective is to determine the feasibility of constructing a two-course pavement system consisting of a lean concrete base and a thin concrete surface wearing course, is located in the southbound lanes of U.S. 41 between Punta Gorda and Ft. Meyers.[38,39] Pavement surfaces include AC, JPCP, CRCP, and fiber-reinforced concrete (FRC), with various joint designs and load transfer mechanisms. While this project originally contained 33 test sections when constructed in 1978, 3 sections that contain an AC surface and 11 sections (several of the CRCP designs and most of the FRC) failed early; thus, only 19 test sections are available for evaluation under this study.

The design matrix for the sections included in the study is given in table 72. All transverse joints are sealed with a preformed compression sealant. The FRC section contains 1-in (25-mm) steel fibers at 158 lb/yd^3 (93.5 kg/m^3). The reinforcement for the CRCP sections consists of either No. 3 or No. 4 bars spaced at varying intervals (all CRCP sections constructed with welded wire fabric reinforcement have been overlaid). In addition, for the CRCP and FRC sections, joints were installed in the underlying LCB at varying intervals.

With the exception of FL 4-1 (representative of Florida's then-standard concrete pavement design), all sections in the project are two-layer structures consisting of a lean concrete base and a thin (2 or 3 in [51 or 76 mm]) concrete surface layer. Three different lean concrete base types were used in this study, each with a different compressive strength:[38,39]

Table 72. Experimental design matrix for selected sections on FL 4.

	3-in (76-mm) JPCP* 15-ft (4.6-m) Skewed Jts		3-in (76-mm) JPCP* 15-ft (4.6-m) Jts		3-in (76-mm) JPCP* 20-ft (6.1-m) Jts		*Control* 9-in (229-mm) JPCP 20-ft (6.1-m) Jts		3-in (76-mm) CRCP*	2-in (51-mm) FRC*
	6-in (152-mm) Stabilized Subgrade		6-in (152-mm) Cement-Treated Subgrade		6-in (152-mm) Stabilized Subgrade		6-in (152-mm) Cement-Treated Subgrade		6-in (152-mm) Stabilized Subgrade	6-in (152-mm) Stabilized Subgrade
	Dowels	No Dowels	Dowels	No Dowels	1 in (25 mm) Dowels	No Dowels	Dowels	No Dowels		
9-in (229-mm) LCB "A" $2000\ lb/in^2$ **(14 MPa)**		4-2		4-7	4-10					
9-in (229-mm) LCB "B" $1250\ lb/in^2$ **(8.6 MPa)**		4-3 4-4[1] 4-5[1]		4-8	4-11				4-12[2] *(0.54% steel)* 4-13[3] *(1.10% steel)* 4-16[3] *(0.30% steel)* 4-14[4] *(0.82% steel)* 4-17[4] *(0.30% steel)* 4-15[5] *(0.67% steel)* 4-18[5] *(0.30% steel)*	4-19[6]
9-in (229-mm) LCB "C" $750\ lb/in^2$ **(5.2 MPa)**		4-6		4-9						
No Base								4-1		

* These sections are bonded to the underlying LCB.
[1] Contains tied lean concrete shoulders.
[2] Joints in base spaced at 20-ft (6.1-m) intervals.
[3] Joints in base spaced at 12-ft (3.7-m) intervals.
[4] Joints in base spaced at 9.33-ft (2.8-m) intervals.
[5] Joints in base spaced at 8-ft (2.4-m) intervals.
[6] Joints in base spaced at 15-ft (4.6-m) intervals.

- LCB Type A — 2,000 lb/in^2 (14.0 MPa)
- LCB Type B — 1,250 lb/in^2 (8.6 MPa)
- LCB Type C — 750 lb/in^2 (5.2 MPa)

The corresponding cement contents for the three different base types are 340, 290, and 220 lb/yd^3 (200, 170, and 130 kg/m^3).

Other variables that can be evaluated under this study are joint spacing (15 ft [4.6 m] vs. 20 ft [6.1 m]), load transfer, pavement type, and type of subgrade stabilization. The "stabilized subgrade" is stabilized with crushed shell to make it dense and uniform. In addition, tied lean concrete shoulders are provided on test sections FL 4-4 and 4-5, which are tied to the concrete surface course on the mainline pavement using No. 4 bars (0.5 in [13 mm]) at 5-ft (1.5-m) spacings. Finally, the effect of different amounts of reinforcing steel and the effect of joints in the underlying LCB can be investigated for the composite CRCP designs.

Traffic Loadings

This four-lane divided roadway currently carries a two-way ADT of 11,600 vehicles per day, including 12 percent heavy trucks. Since opening to traffic in 1978, this pavement has accumulated 4.5 million 18-kip (80 kN) ESAL applications in the outer traffic lane.

Performance

The outer lane performance data for the FL 4 sections are summarized in table 73. This information is portrayed in graphical format in figure 71 for the JPCP sections and in figure 72 for the CRCP and FRC sections.

JPCP Sections

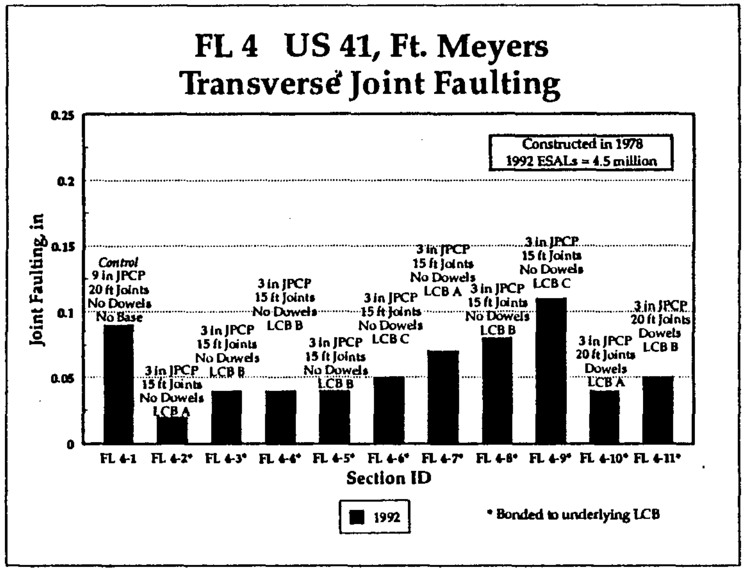
Generally, the JPCP sections constructed on a higher strength LCB provided slightly better performance than those constructed on a lower strength LCB. The sections on higher strength LCB had slightly less faulting and cracking than the other sections. Nevertheless, the performance of one section constructed on the lowest strength LCB (FL 4-6) was comparable to the corresponding section constructed on the highest strength LCB (FL 4-2).

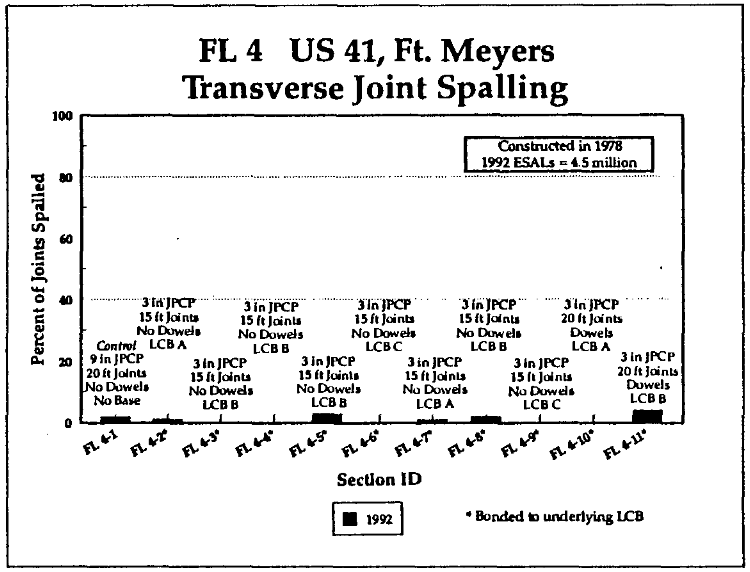
Sections constructed on stabilized subgrades (FL 4-2 through 4-6) appear to be performing better than those on cement-treated subgrades (FL 4-7 through 4-9), although this effect is confounded by joint orientation. Sections constructed on stabilized subgrades exhibit lower levels of faulting and cracking than those constructed on the cement-treated subgrade. However, the skewed joint design also may be contributing to the lower faulting observed on the stabilized sections.

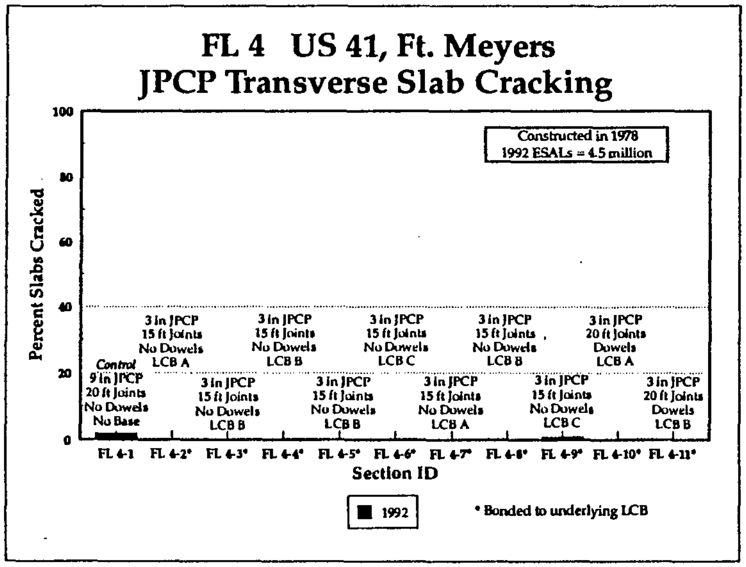
Table 73. Summary of 1992 outer lane performance data for FL 4.

		3-in (76-mm) JPCP* 15-ft (4.6-m) Skewed Jts			**3-in (76-mm) JPCP* 15-ft (4.6-m) Jts**	**3-in (76-mm) JPCP* 20-ft (6.1-m) Jts**	***Control,* 9-in (229-mm) JPCP 20-ft (6.1-m) Jts**	**3-in (76-mm) CRCP***							**2-in (51-mm) FRC***
		6-in (152-mm) Stabilized Subgrade			**6-in (152-mm) Cement-Treated Subgrade**	**6-in (152-mm) Stabilized Subgrade**	**6-in (152-mm) Cement-Treated Subgrade**	**6-in (152-mm) Stabilized Subgrade**							**6-in (152-mm) Stabilized Subgrade**
		No Dowels			No Dowels	1 in (25 mm) Dowels	No Dowels								
Age at Survey, years		14			14	14	14	14							14
ESAL's, millions		4.5			4.5	4.5	4.5	4.5							4.5
9-in (229-mm) LCB "A" *2000 lb/in^2* (14 MPa)		4-2			4-7	4-10									
	Faulting, in	0.02			0.07	0.04									
	Det. Tr. Crks/mi	0			0	0									
	% Slabs Cracked	0			0	0									
	Long. Cracks, ft/mi	0			0	0									
	% Joints Spalled	1			1	0									
	PSR	3.8			3.7	3.6									
	IRI, in/mi	139			110	112									
9-in (229-mm) LCB "B" *1250 lb/in^2* (8.6 MPa)		4-3	4-4	4-5	4-8	4-11		4-12	4-13	4-16	4-14	4-17	4-15	4-18	4-19
	Faulting, in	0.04	0.04	0.04	0.08	0.05		—	—	—	—	—	—	—	—
	Det. Tr. Crks/mi	0	0	0	0	0		30	103	453	107	439	85	295	245
	% Slabs Cracked	0	0	0	0	0		—	—	—	—	—	—	—	—
	Long. Cracks, ft/mi	546	0	0	0	0		0	105	0	406	0	132	553	2955
	% Jts Spalled	0	0	3	0	4		—	—	—	—	—	—	—	—
	Failures/mi (CRC/FRC)	—	—	—	—	—		0	0	0	0	0	0	0	0
	PSR	3.7	3.9	3.6	3.6	3.6		—	—	—	—	—	—	—	—
	IRI, in/mi	122	98	110	106	116		—	—	—	—	—	—	—	—
9-in (229-mm) LCB "C" *750 lb/in^2* (5.2 MPa)		4-6			4-9										
	Faulting, in	0.05			0.11										
	Det. Tr. Crks/mi	0			5										
	% Slabs Cracked	0			1										
	Long. Cracks, ft/mi	0			1513										
	% Joints Spalled	0			0										
	PSR	3.7			3.2										
	IRI, in/mi	95			125										
No Base							4-1								
	Faulting, in						0.09								
	Det. Tr. Crks/mi						5								
	% Slabs Cracked						2								
	Long. Cracks, ft/mi						0								
	% Joints Spalled						2								
	PSR						3.6								
	IRI, in/mi						143								

* These sections are bonded to the underlying LCB.
1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km







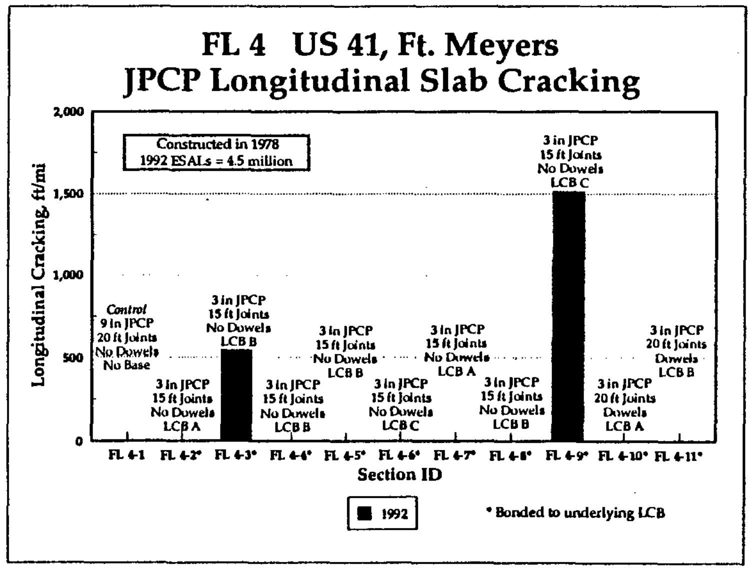
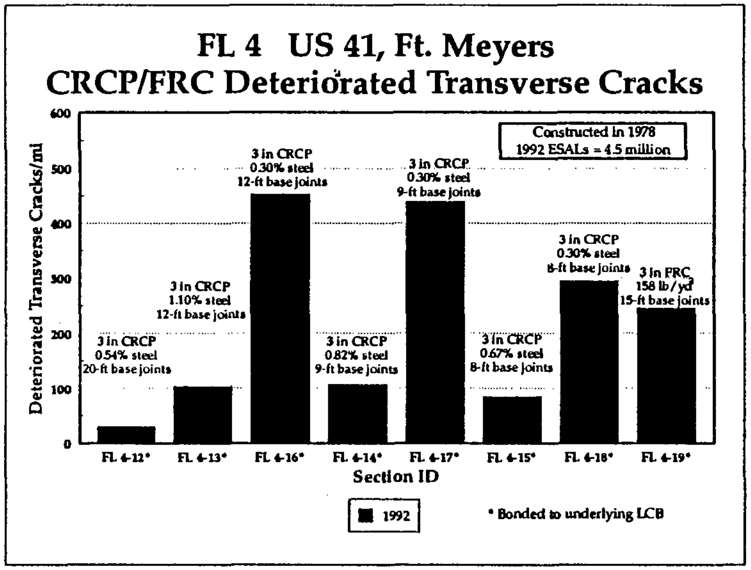
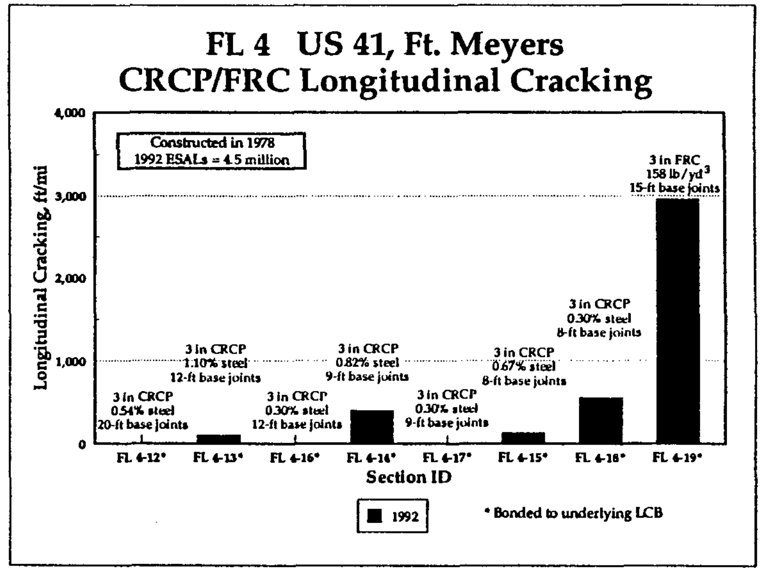


Figure 71. Performance summary for JPCP sections for FL 4.





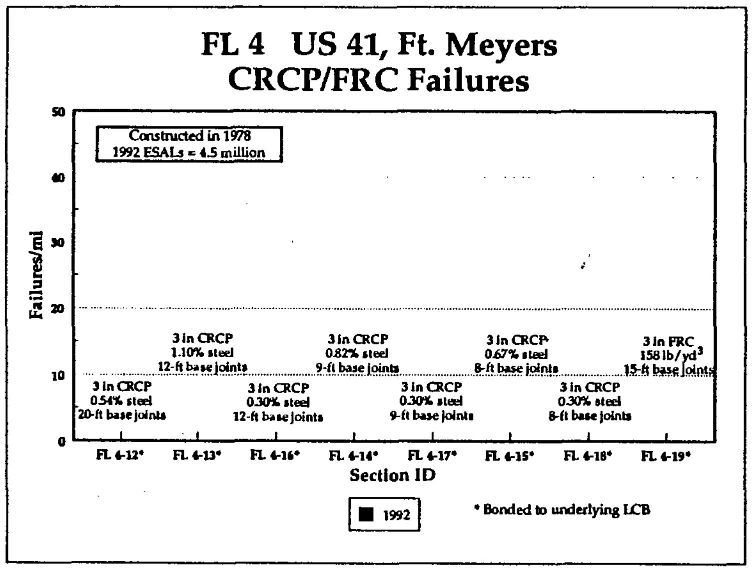


Figure 72. Performance summary for CRCP/FRC sections for FL 4.

Sections 4-4 and 4-5 each contained a tied PCC shoulder. Although these sections are performing very well with no cracking and nominal amounts of spalling and faulting, their performance at this time is not significantly different from Fl 4-2, 4-3, or 4-6.

While direct comparisons of the effect of joint spacing are not possible, the small amount of cracking that developed on the 20-ft (6.1-m) slabs is noteworthy. The only 20-ft (6.1-m) section that exhibited transverse cracking was the standard design (FL 4-1). No appreciable difference in spalling is evident between the 15- and 20-ft (4.6-6.1-m) sections, and the preformed compression sealant placed in the transverse joints were in excellent condition.

At this time, the advantages of dowel bars in reducing faulting are not clear. The faulting of the doweled sections (4-10 and 4-11) is actually slightly higher than the faulting of the corresponding nondoweled sections on stabilized base (4-2, 4-3, 4-4, and 4-5), although the difference is not significant. The doweled sections are less faulted than the standard section (4-1) and the composite sections placed on cement-treated subgrade (4-7, 4-8, and 4-9).

Longitudinal cracking was identified on two JPCP sections (FL 4-3 and 4-9). The cracking on FL 4-3 occurred in the center of the inner traffic lane, while the majority of the longitudinal cracking on FL 4-9 occurred in the outer traffic lane near the centerline joint.

The performance data indicate that the two-layer concrete slab systems can provide performance comparable or superior to that of a monolithic structure. For example, FL 4-7 (constructed on an LCB with 2000 lb/in^2 [14.0 MPa] compressive strength) and FL 4-8 (constructed on an LCB with 1250 lb/in^2 [8.6 MPa] compressive strength) provide performance similar to (or perhaps even slightly better than) the monolithic control section (FL 4-1). However, the performance of FL 4-9 (constructed on an LCB with 750 lb/in^2 [5.2 MPa] compressive strength) is performing worse than the monolithic control section.

The Florida Department of Transportation has been monitoring these sections since their construction. A recent report indicates that the best performing sections are the composite designs placed on stabilized bases with 15-ft (4.6-m) skewed joints (FL 4-2 through 4-6).[39]

CRCP/FRC Sections

Performance data were collected for the seven remaining CRCP designs (constructed with a range of steel contents and varying base joint spacings) and the lone remaining FRC design. Again, these designs are composite designs consisting of a thin PCC surface over an LCB. Unfortunately, roughness and serviceability data were not collected for these sections.

Figure 72 shows that the CRCP sections with the smallest amount of reinforcing steel (FL 4-16, 4-17, and 4-18) all show the greatest amount of deteriorated transverse cracking, much higher than any of the other CRCP sections. The FRC section also exhibits more deteriorated transverse cracking than the other sections. The section with only 0.54 percent reinforcement shows the least amount of deteriorated transverse cracking; the sections with 0.67 percent, 0.82 percent, and 1.10 percent steel all exhibit about the same amount of deteriorated transverse cracking. The transverse cracking for these latter sections has exceeded the critical levels for continuously reinforced concrete pavements.

The effect of the spacing of the joints in the LCB is not clear. Some evidence suggests that shorter base joint spacings reduce the amount of deteriorated cracking (compare FL 4-16, 4-17, and 4-18), yet the section with the least amount of transverse cracking (FL 4-12) has base joints spaced at 20-ft (6.1-m) intervals.

Three sections are exhibiting significant amounts of longitudinal cracking: FL 4-14, 4-18, and 4-19. In particular, FL 4-19 (the FRC design) displays longitudinal cracking over about 60 percent of its total length. Most of this cracking occurs near the pavement centerline, suggesting problems due to late or inadequate sawing of the longitudinal joint. To their credit, however, none of the CRCP or FRC designs show any failures (punchouts).

Roughness and Serviceability

Figure 73 shows the roughness and serviceability data for the JPCP sections on FL 4. Unfortunately, that data were not collected for the CRCP or FRC sections, so the roughness of the various designs cannot be compared.

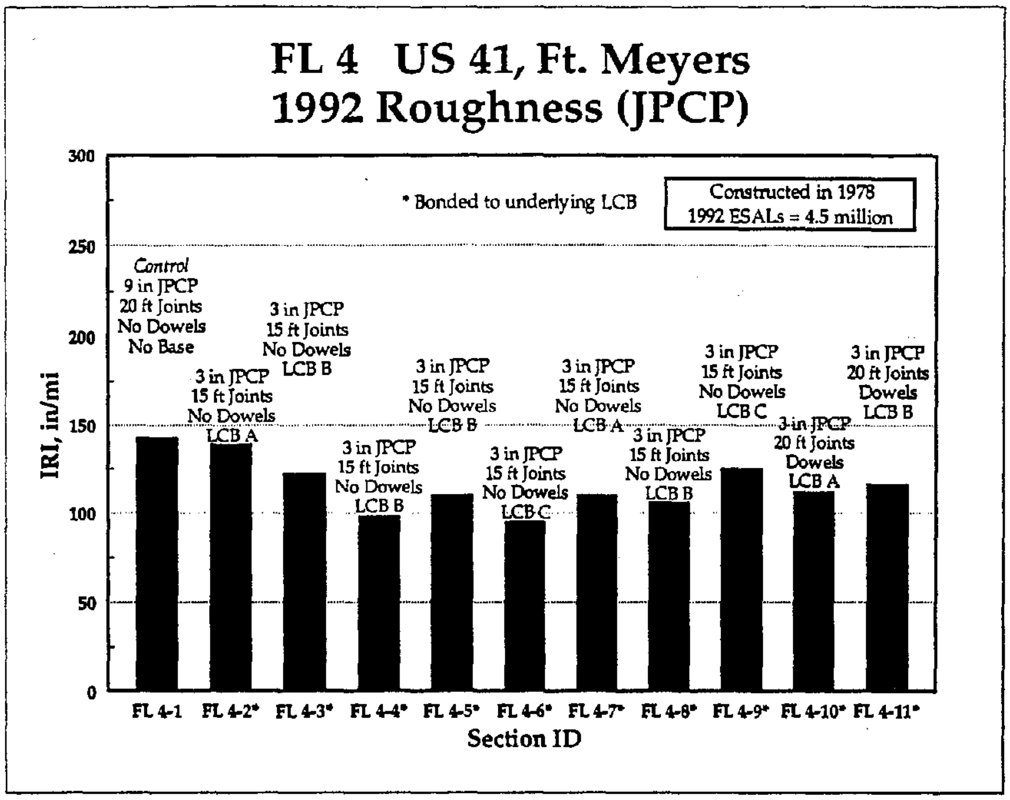
None of the JPCP sections are exceedingly rough, although two sections (FL 4-1 and 4-2) are rougher than the others. FL 4-1 had a significant amount of faulting, but it is somewhat surprising that FL 4-2 is as rough as it is, given the minimal amount of faulting that has developed on that section. FL 4-9, although it also has a high roughness level, had the lowest serviceability rating of the JPCP sections. The smoothest JPCP sections are FL 4-4 and 4-6, each of which are showing roughness levels below 100 in/mi (1.6 m/km).

Summary

The performance of the FL 4 sections is summarized below:

- The JPCP sections constructed on a higher strength LCB generally provided slightly better performance than those constructed on a lower strength LCB, although the performance levels are not considered significantly different.

- Sections constructed on shell-stabilized subgrades appear to be performing better than those on cement-treated subgrades (FL 4-7 through 4-9). However, this effect is confounded by joint orientation.



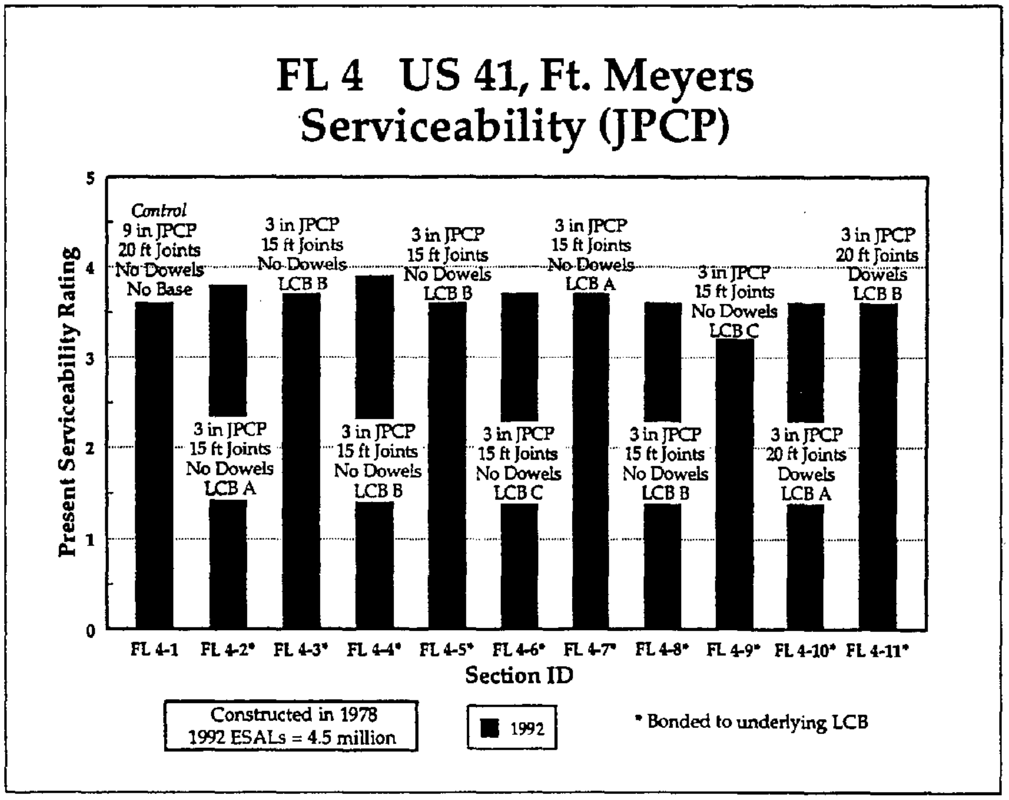


Figure 73. Roughness and serviceability for JPCP sections for FL 4.

- At this time, no apparent performance benefits are being derived from the tied PCC shoulders. Although the two sections with tied PCC shoulders are performing well, their performance is not significantly different from the corresponding sections with AC shoulders.

- The effect of joint spacing on performance cannot be established because of confounding factors. However, the small amount of cracking that developed on the 20-ft (6.1-m) slabs is noteworthy.

- Very little spalling was observed on this project, the result of perhaps the mild climate and the effectiveness of the preformed compression seals.

- At this time, the advantages of dowel bars in reducing faulting are ambiguous. The faulting of the doweled sections is actually slightly higher than the faulting of the corresponding nondoweled sections on stabilized base, although the difference is not significant.

- The CRCP composite sections with the smallest amount of reinforcing steel displayed the greatest amount of deteriorated transverse cracking. The FRC section also exhibited a substantial amount of deteriorated transverse cracking. The section with only 0.54 percent reinforcement shows the least amount of deteriorated transverse cracking, even less than those sections containing 0.67 percent, 0.82 percent, and 1.10 percent steel.

- On the CRCP composite sections, the effect of the spacing of the joints in the LCB is not clear. Some evidence suggests that shorter base joint spacing reduces the amount of deteriorated cracking, although the section with the least amount of transverse cracking has base joints spaced at 20-ft (6.1-m) intervals.

- None of the CRCP or FRC designs are exhibiting any punchout failures, although three sections are displaying significant amounts of longitudinal cracking. Some of this cracking occurred near the pavement centerline, suggesting that inadequate joint sawing practices may be responsible.

- None of the JPCP sections are exceedingly rough. The smoothest riding sections are those constructed on stabilized base with 15-ft (4.6-m) skewed joints.

- Overall, the performance data from the FL 4 indicate that the two-layer concrete slab systems can provide performance comparable or superior to that of a monolithic structure. In particular, the sections constructed on stabilized subgrade with 15-ft (4.6-m) skewed joints are performing better than the standard 9-in (229-mm) monolithic section.

Georgia 1 and Georgia 2 (I-85, Newnan and La Grange, Georgia)

In 1971, an experimental project was constructed by the Georgia Department of Transportation in the southbound lanes of I-85 near Newnan. Consisting of 10 different test sections, the purpose of this project is to evaluate the effects of dowels and base type on the performance of JPCP.[40]

All 10 GA 1 sections are 9-in (229-mm) JPCP with 20-ft (6.1-m) skewed joints that are sealed with a hot-poured sealant material. The base types include a 1-in (25-mm) AC layer over a 5-in (127-mm) CTB, a 6-in (152-mm) CTB, and a 4-in (102-mm) ATB. Dowels, when included, were 1.13 in (29 mm) in diameter, 18 in (457 mm) long, spaced at 15-in (381-mm) intervals, and coated with red lead paint. Replicate sets of sections were also constructed for all base types, except for the 6-in (127-mm) CTB section, which represented the standard DOT design at the time. All GA 1 sections were constructed on an AASHTO A-4 subgrade. The experimental design matrix for this project is shown in table 74. These experimental sections were diamond ground in 1985. The reason for diamond grinding is believed to be due to high roughness levels after initial construction.

Table 74. Experimental design matrix for GA 1 and GA 2.

<table>
<tr><th></th><th colspan="2">9-in (229-mm) JPCP
20-ft (6.1-m) Skewed Joints</th><th>9-in (229-mm) JPCP
20-ft (6.1-m) Joints</th></tr>
<tr><th></th><th>1.12-in (29-mm)
Dowel Bars</th><th>No
Dowels</th><th>1.12-in (29-mm)
Dowel Bars</th></tr>
<tr><td>1-in (25-mm) AC (4-5% AC-20)
5-in (127-mm) CTB
(6% cement)</td><td>GA 1-1
GA 1-3</td><td>GA 1-2
GA 1-4</td><td></td></tr>
<tr><td>6-in (152-mm) CTB
(6% cement)</td><td>GA 1-5</td><td>GA 1-10
(original control)</td><td></td></tr>
<tr><td>4-in (102-mm) ATB
(4.5% AC-20)</td><td>GA 1-6
GA 1-8</td><td>GA 1-7
GA 1-9</td><td></td></tr>
<tr><td>6-in (152-mm) LCB
(6.9% cement)</td><td></td><td></td><td>GA 2
(new control)</td></tr>
</table>

GA 1-1, 1-2, 1-3, and 1-4 have 5-in (127-mm) cement-treated subbase.
All other sections have no subbase.

An adjacent concrete pavement section, GA 2, was included in this evaluation because it contains a lean concrete base. GA 2, located about 20 mi (32 km) south of GA 1 in the southbound lanes of I-85, was constructed in 1977. It is also a 9-in (229-mm) JPCP with 20-ft (6.1-m) perpendicular joints and contains 1.13-in (29-mm) diameter dowel bars. It was constructed on an AASHTO A-2-6 subgrade. Unlike the GA 1 sections, GA 2 has not been diamond ground.

Traffic Loadings

The GA 1 sections consist of a four-lane divided roadway that currently carries a two-way ADT of 40,000 vehicles per day, including 16 percent heavy trucks. At the time that diamond grinding was conducted in 1985, the pavement had sustained 12.6 million ESAL applications in the outer traffic lane; since the diamond grinding and through 1992, the pavement has sustained 6.5 million ESAL applications in the outer traffic lane.

The GA 2 section has sustained 12.1 million ESAL applications in the outer traffic lane through 1992. The section currently carries a two-way ADT of 32,000 vehicles per day, of which 16 percent are heavy trucks.

Performance

A summary of the outer lane performance data for these sections is shown in table 75. This performance data is displayed graphically in figure 74. The performance data indicate that all sections are in excellent condition. None of the sections display any transverse or longitudinal cracking, faulting levels are minimal (recall that the GA 1 sections were ground in 1985), and a small amount of joint spalling is present on only one section.

Because of the absence of distress on these sections, it is difficult to draw definite conclusions regarding the effect of dowel bars and the different base types on pavement performance. The absence of transverse cracking on the 20-ft (6.1-m) slabs is particularly noteworthy, given the age of the pavements and the traffic levels that it has endured.

The low level of faulting is no doubt due to the diamond grinding that was conducted in 1985. Since that time, some joint faulting has re-developed. Based on the current data, the doweled sections do appear to have slightly less faulting than the nondoweled sections, although the difference is not significant. In addition, the sections constructed on CTB are showing the most faulting and the sections placed on ATB appear to be most resistant to faulting. Similar observations were made by the Georgia Department of Transportation in a 1982 memo that indicated that the sections constructed on ATB and on the AC/CTB had less faulting than the sections placed on CTB.[41]

Roughness and Serviceability

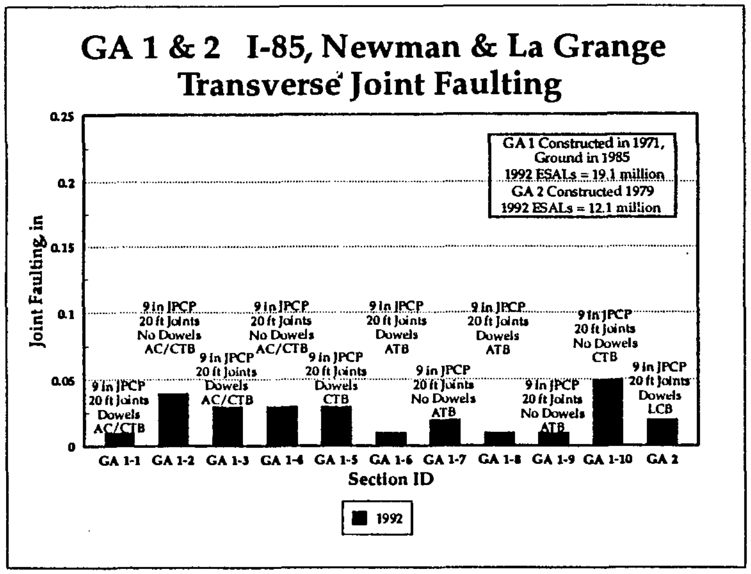
The roughness and serviceability data for the GA 1 and GA 2 sections are presented in figure 75. All GA 1 sections are extremely smooth, even though it has been 7 years since they had been diamond ground. Because of the low roughness levels, no trends in the smoothness of different designs are apparent.

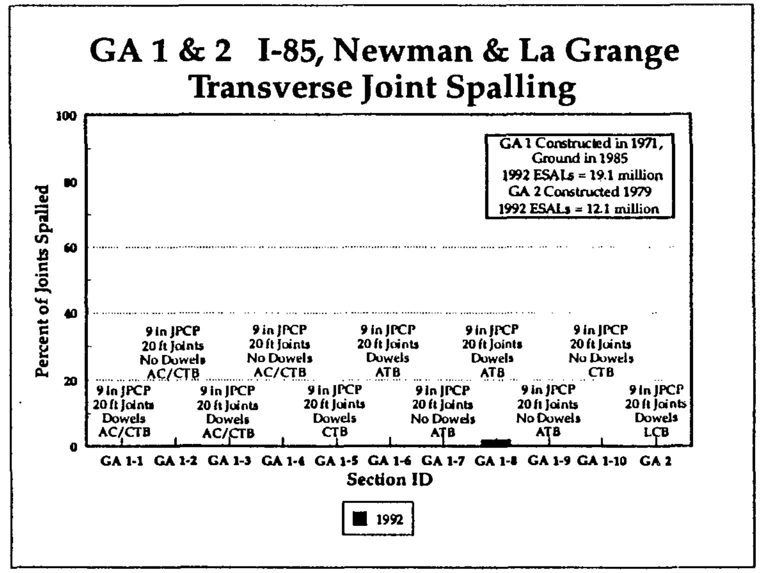
Table 75. Summary of 1992 outer lane performance data for Georgia sections.

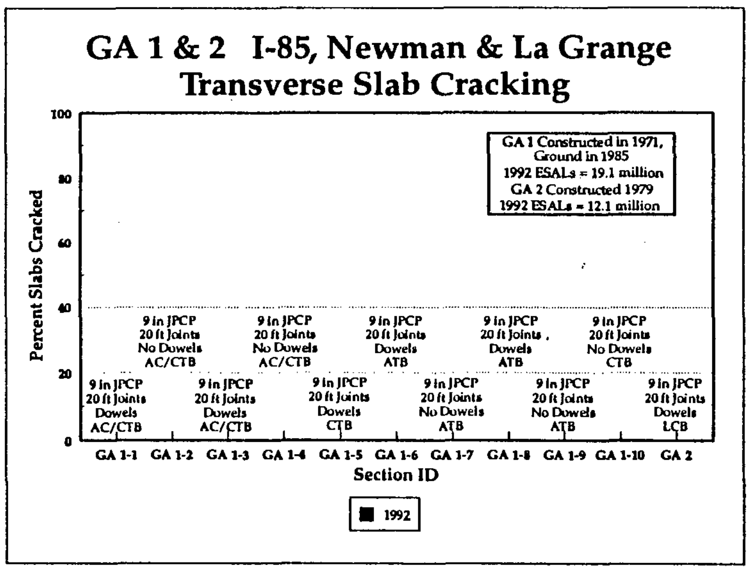
		9-in (229-mm) JPCP 20-ft (6.1-m) Skewed Joints				9-in (229-mm) JPCP 20-ft (6.1-m) Joints
		1.12-in (29-mm) Dowel Bars		No Dowels		1.12-in (29-mm) Dowel Bars
		1992 Survey	1992 Rep	1992 Survey	1992 Rep	1992 Survey
Age at Survey, years		21	21	21	21	15
ESAL's, 1971–1985, millions		12.6	12.6	12.6	12.6	—
ESAL's, 1985–1992, millions		6.5	6.5	6.5	6.5	—
Total ESAL's, millions		19.1	19.1	19.1	19.1	12.1
1-in (25-mm) AC (4.5% AC-20) 5-in (127-mm) CTB (6% cement)		GA 1-1	GA 1-3	GA 1-2	GA 1-4	
	Faulting, in	0.01	0.03	0.04	0.03	
	Det. Tr. Crks/mi	0	0	0	0	
	% Slabs Cracked	0	0	0	0	
	Long. Cracks, ft/mi	0	0	0	0	
	% Joints Spalled	0	0	0	0	
	PSR	4.1	4.1	4.1	4.1	
	IRI, in/mi	60	54	70	51	
6-in (152-mm) CTB (6% cement)		GA 1-5		GA 1-10		
	Faulting, in	0.03		0.05		
	Det. Tr. Crks/mi	0		0		
	% Slabs Cracked	0		0		
	Long. Cracks, ft/mi	0		0		
	% Joints Spalled	0		0		
	PSR	4.0		4.0		
	IRI, in/mi	51		54		
4-in (102-mm) ATB (4.5% AC-20)		GA 1-6	GA 1-8	GA 1-7	GA 1-9	
	Faulting, in	0.01	0.01	0.02	0.01	
	Det. Tr. Crks/mi	0	0	0	0	
	% Slabs Cracked	0	0	0	0	
	Long. Cracks, ft/mi	0	0	0	0	
	% Joints Spalled	0	2	0	0	
	PSR	4.1	4.1	4.0	4.1	
	IRI, in/mi	43	50	49	56	
6-in (152-mm) LCB (6.9% cement)						GA 2
	Faulting, in					0.02
	Det. Tr. Crks/mi					0
	% Slabs Cracked					0
	Long. Cracks, ft/mi					0
	% Joints Spalled					0
	PSR					3.6
	IRI, in/mi					107

All GA 1 sections were diamond ground in 1985.
GA 1-1, 1-2, 1-3, and 1-4 have 5-in cement-treated subbase.
All other sections have no subbase.

1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km







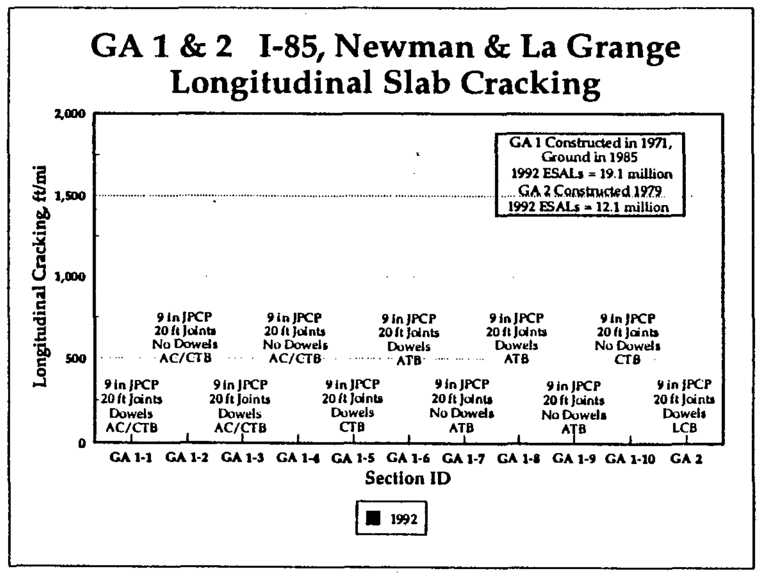
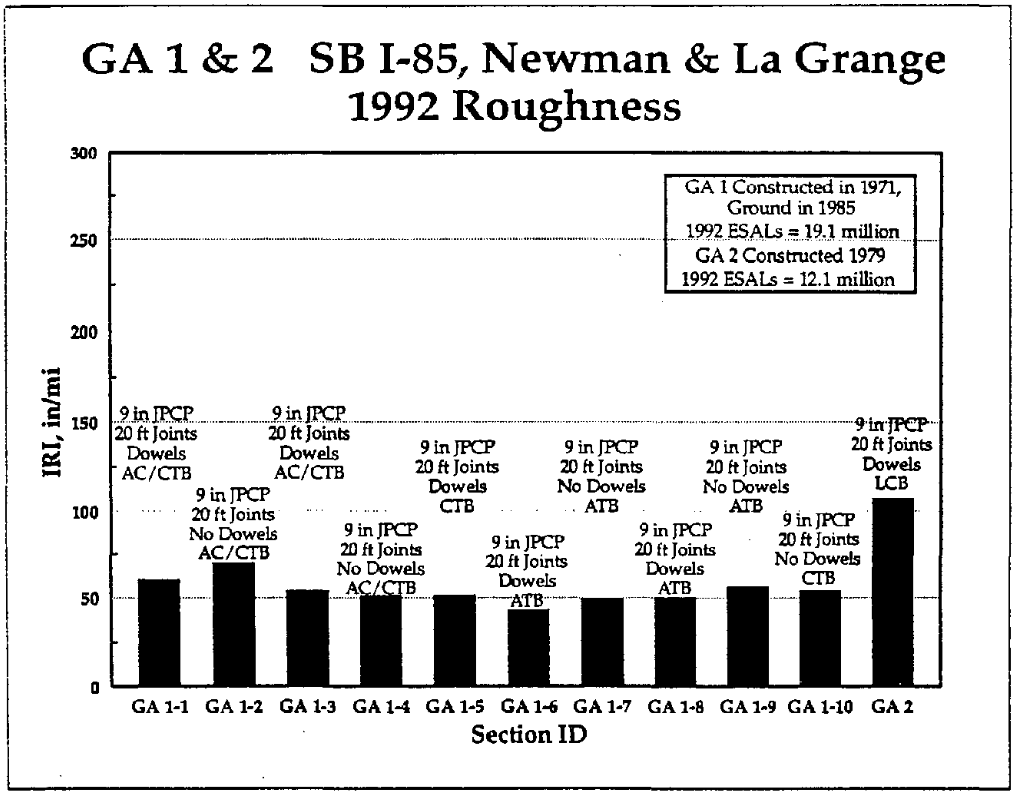


Figure 74. Performance summary for GA 1 and GA 2.



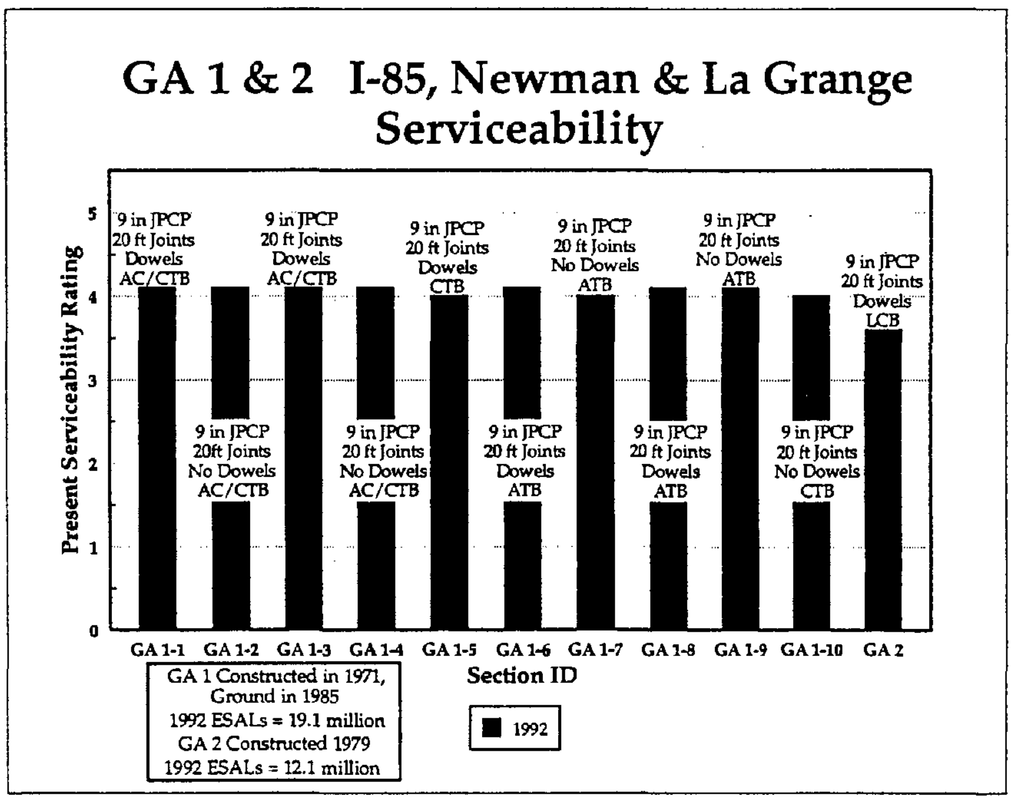


Figure 75. Roughness and serviceability for GA 1 and GA 2.

Figure 75 shows that GA 2, which was constructed in 1977 and has not been diamond ground, is rougher than the GA 1 sections. However, its overall roughness level is still quite acceptable.

Summary

The performance of the GA 1 and GA 2 sections is summarized as follows:

- All sections are in excellent condition, which makes the evaluation of the different designs difficult. No section exhibited any cracking, faulting levels were low, and only one section exhibited any joint spalling. The absence of transverse cracking on these 20-ft (6.1-m) JPCP sections is considered noteworthy.

- The GA 1 sections had been diamond ground in 1985, and some faulting has begun to re-develop. Based on the current data, the doweled sections do appear to have slightly less faulting than the nondoweled sections, although the difference is not significant. In addition, the sections constructed on CTB are showing the most faulting and the sections placed on ATB appear to be most resistant to faulting.

- The roughness and serviceability data indicate that all GA 1 sections are extremely smooth. Because of the low roughness levels, no trends in the smoothness values of the different designs are apparent. GA 2 is rougher than the GA 1 sections, but its overall roughness level is still low.

North Carolina 1

Several experimental pavement sections were constructed on I-95 near Rocky Mount, North Carolina, in 1967.[42,43] Design variables in the project include base type, pavement type, joint spacing, slab thickness, joint skew, and load transfer. The subgrade soil for these sections varies from AASHTO A-4 to A-7-6. The design matrix is depicted in table 76, which shows that an adjacent CRCP section is also included as part of the study.

Traffic Loadings

This four-lane, rural Interstate roadway carries a two-way ADT of 29,700 vehicles per day, including 20 percent heavy trucks. In 1987, the outer traffic lane had accumulated 9.0 million ESAL application; through 1992, the pavement has sustained 16.0 million ESAL applications.

Performance

Table 77 provides the outer lane performance data for the NC 1 sections. These data are portrayed in graphical format in figure 76.

Table 76. Experimental design matrix for NC 1.

	9-in (229-mm) JPCP 30-ft (9.1-m) Jts			8-in (203-mm) JRCP 0.17% Steel 60-ft (18.3-m) Jts	8-in (203-mm) CRCP 0.6% Steel
	Skewed Joints	Perpendicular Joints		Perpendicular Joints	
	No Dowels	1-in (25-mm) Dowels	No Dowels	1-in (25-mm) Dowels	
4-in (102-mm) AGG	NC 1-1	NC 1-4	NC 1-8	NC 1-7	NC 1-9
6-in (152-mm) Soil Cement (8% cement)		NC 1-2	NC 1-3		
4-in (102-mm) CTB (6% cement)			NC 1-5		
4-in (102-mm) ATB (4% AC-20)			NC 1-6		

All sections are daylighted.

Of the jointed concrete sections, NC 1-6 (9-in [229-mm] JPCP constructed on an ATB) is providing the best performance. The faulting on this section is the lowest of all sections, even though it does not contain dowel bars. This observation is particularly intriguing, as the ATB apparently was able to resist faulting better than sections placed on either of the cementitious bases, and those that contained dowel bars. In addition, NC 1-6 is displaying no spalling or longitudinal cracking, and shows only a small amount of transverse cracking.

The sections constructed on cementitious bases were not only susceptible to substantial joint faulting, they also exhibit a significant amount of longitudinal cracking. In particular, the soil cement sections displayed an exorbitant amount of cracking that may be related to the amount of cement added to the mix. Although the specifications called for 6 percent cement, post-construction testing indicated that 8 percent had actually been added.[42] This amount of cement probably made the base stiffer, which could contribute to increased curling stresses and the subsequent development of the longitudinal cracking. The section placed on CTB, although not displaying as much longitudinal cracking as the sections on soil cement, still exhibits a significant amount of longitudinal cracking.

The pavement sections constructed on aggregate base display significant joint faulting, even those that contain dowel bars. Three of these four sections also exhibit varying amounts of transverse cracking, with one section (NC 1-8) showing 77 percent of the slabs cracked. However, the long joint spacing (30 ft [9.1 m]) may contribute to transverse cracking. Because none of the other sections placed on an aggregate base exhibit that much cracking, it is believed that a factor other than

Table 77. Summary of 1987 and 1992 outer lane performance data for NC 1.

		9-in (229-mm) JPCP 30-ft (9.1-m) Jts						8-in (203-mm) JRCP 0.17% Steel 60-ft (18.3-m) Jts		8-in (203-mm) CRCP 0.6% Steel
		Skewed Joints		Perpendicular Joints				Perp. Joints		
		No Dowels		1-in (25-mm) Dowels		No Dowels		1-in (25-mm) Dowels		
		1987	1992	1987	1992	1987	1992	1987	1992	1992
Age at Survey, years		20	25	20	25	20	25	20	25	25
ESAL's, millions		9.0	16.0	9.0	16.0	9.0	16.0	9.0	16.0	16.0
4-in (102-mm) AGG		1-1	1-1	1-4	1-4	1-8	1-8	1-7	1-7	1-9
	Faulting, in	0.12	0.13	0.13	0.13	0.22	0.23	0.15	0.15	—
	Det. Tr. Crks/mi	5	20	0	0	64	136	0	0	0
	% Slabs Cracked	3	11	0	0	37	77	—	—	—
	Long. Cracks, ft/mi	0	0	0	20	0	0	295	293	0
	% Joints Spalled	0	0	0	0	0	0	0	11	0
	Fails/mi (CRCP only)	—	—	—	—	—	—	—	—	0
	PSR	3.4	3.3	3.4	3.2	3.7	3.3	3.4	3.2	3.7
	IRI, in/mi	—	111	—	110	—	131	—	120	—
6-in (152-mm) Soil Cement (8% cement)				1-2	1-2	1-3	1-3			
	Faulting, in			0.16	0.16	0.13	0.14			
	Det. Tr. Crks/mi			10	10	5	10			
	% Slabs Cracked			6	6	3	6			
	Long. Cracks, ft/mi			1451	2725	3068	4983			
	% Joints Spalled			0	3	0	3			
	PSR			3.5	3.3	3.6	3.4			
	IRI, in/mi			—	114	—	116			
4-in (102-mm) CTB (6% cement)						1-5	1-5			
	Faulting, in					0.16	0.16			
	Det. Tr. Crks/mi					0	0			
	% Slabs Cracked					0	0			
	Long. Cracks, ft/mi					179	372			
	% Joints Spalled					0	0			
	PSR					3.2	2.9			
	IRI, in/mi					—	139			
4-in (102-mm) ATB (4% AC-20)						1-6	1-6			
	Faulting, in					0.05	0.03			
	Det. Tr. Crks/mi					0	15			
	% Slabs Cracked					0	9			
	Long. Cracks, ft/mi					0	0			
	% Joints Spalled					0	0			
	PSR					3.8	3.7			
	IRI, in/mi					—	102			

All sections are daylighted.

1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km

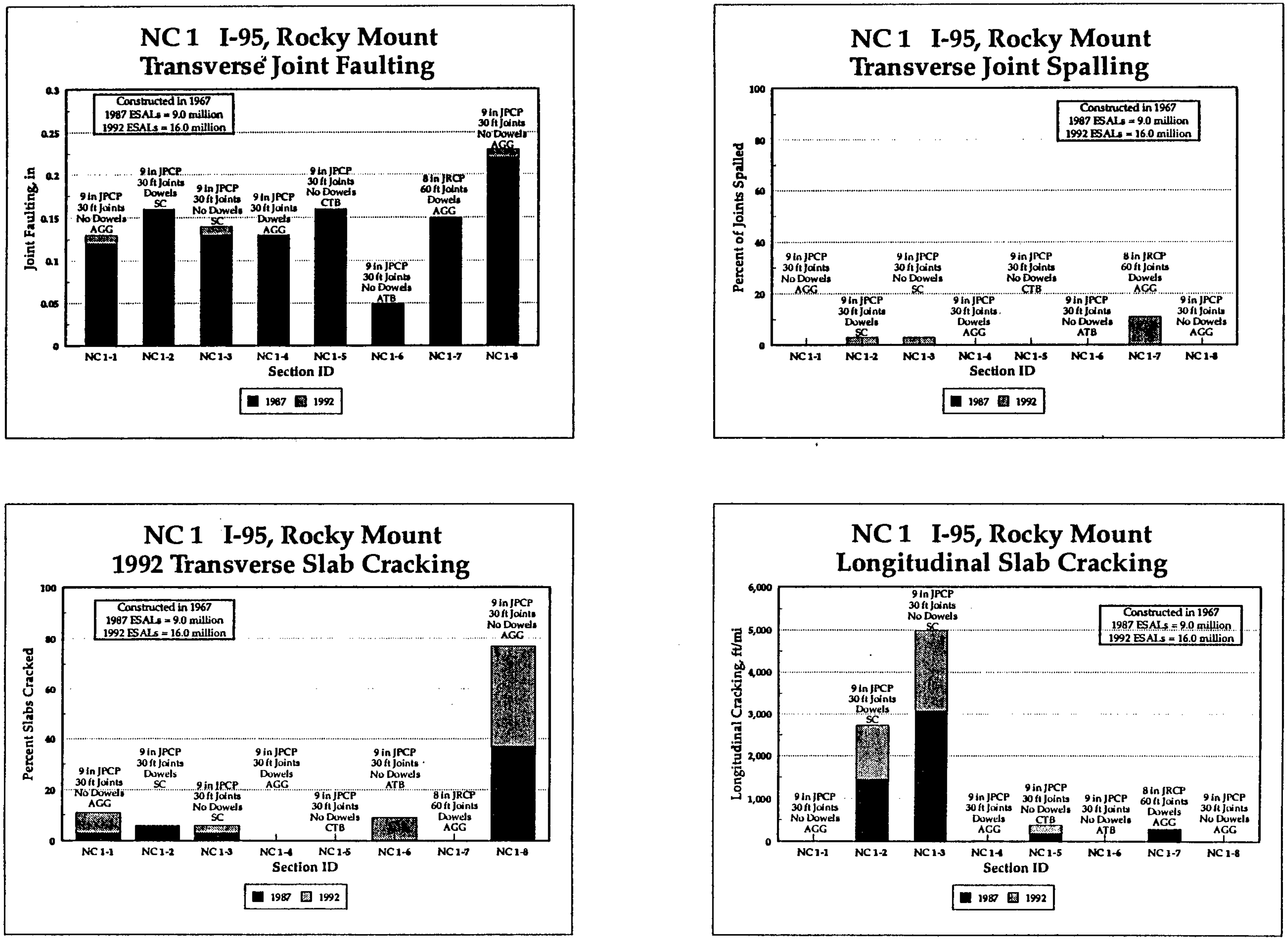


Figure 76. Performance summary for NC 1.

design may be responsible (e.g., poor subgrade compaction or a construction problem).

Comparing the corresponding sections with and without dowel bars, the dowel bars are not effective in reducing the development of joint faulting. The levels of faulting exhibited by the doweled sections is unacceptable and is not significantly different from the nondoweled sections. The inadequacy of the dowel bars in reducing faulting may be attributed to the small size of the dowels (1-in [25-mm] diameter), which is believed to be insufficient for Interstate highways.

The section constructed with skewed transverse joints (NC 1-1) is performing better than the corresponding section with perpendicular joints (NC 1-8). The section with perpendicular joints shows considerably more faulting and transverse cracking, as well as more roughness.

The 8-in (203-mm) JRCP is showing fair to good performance, certainly comparable to several of the 9-in (229-mm) JPCP sections (NC 1-4 or NC 1-5). Even though it is exhibiting a fairly high level of faulting, the JRCP section has not developed any deteriorated transverse cracks. It does, however, show the most joint spalling of any of the sections, and this may be related to the greater joint movements associated with the longer 60-ft (18.3-m) slabs.

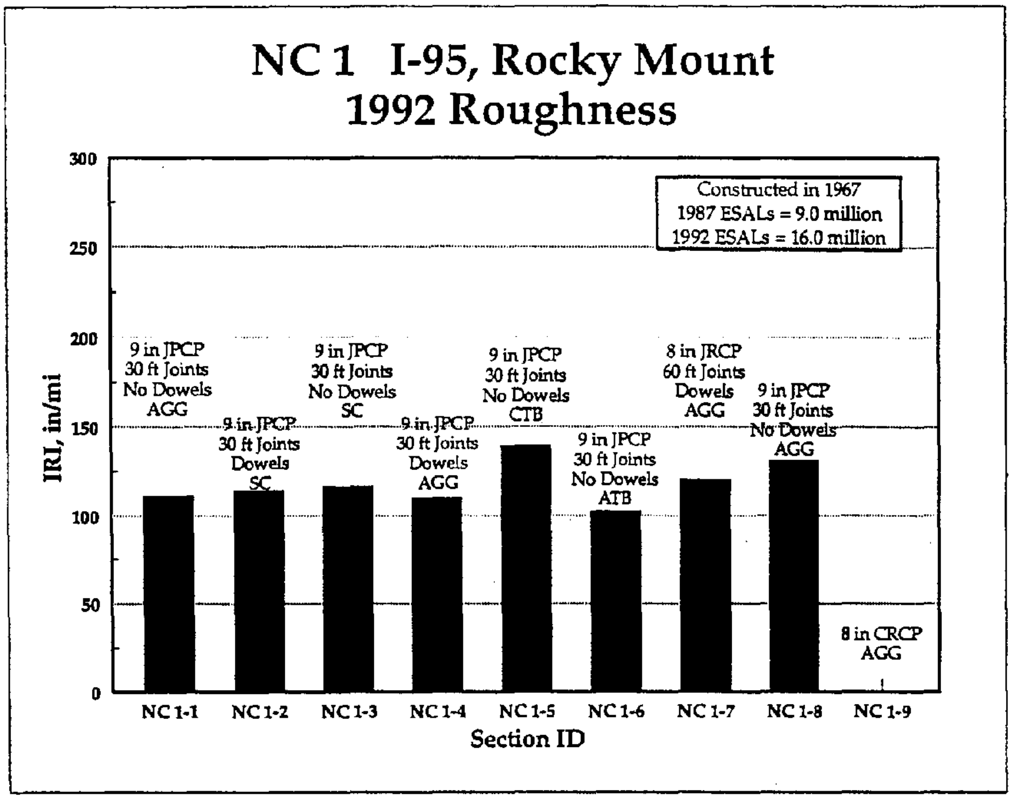
On the topic of joint spalling, that distress is conspicuously absent from these 25-year-old sections. The mild climate and the recently-maintained hot-poured joint sealant (which had hardened and failed in a few instances) apparently are contributing to this performance.

Section 1-9 is the CRCP section (0.60% steel) that was constructed as part of this project. The evaluation of this section indicates that it is performing extremely well, with no deteriorated cracks or pavement failures noted within the survey section. The low severity transverse cracks are spaced at about 4.3-ft (1.3-m) intervals.

Roughness and Serviceability

Figure 77 presents the roughness and serviceability data for the NC 1 sections. With the exception of NC 1-5 and NC 1-8, the roughness for all of the sections appears to be satisfactory. The smoothest riding section is NC 1-6, which was also noted to be performing the best.

All sections are noted to have decreased in serviceability from 1987. Sections 1-5 and 1-8 have low serviceabilities, ones that have reached or are approaching "trigger" values for rehabilitation. Section 1-6 and section 1-9 (the CRCP design) have the highest serviceabilities. The high serviceability of the CRCP section confirms its excellent performance.



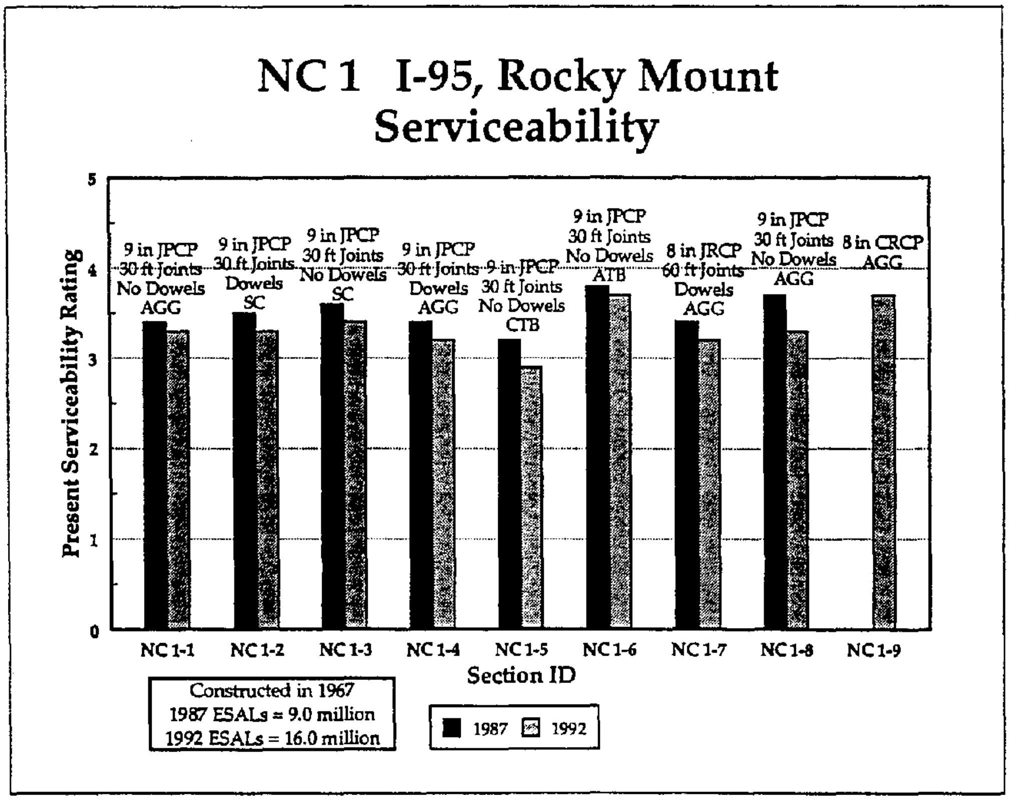


Figure 77. Roughness and serviceability for NC 1.

5-Year Performance Trends

The 5-year performance trends for the NC 1 sections are shown in figure 78. This figure shows that joint faulting and joint spalling have not changed significantly since the 1987 survey. NC 1-8 shows a marked increase in transverse slab cracking, while the other sections show only a small increase, if any.

Longitudinal cracking greatly increased over the 5-year period for the sections constructed on soil cement base (NC 1-2, NC 1-3). No other sections show any appreciable difference in longitudinal cracking.

Summary

The performance of the NC 1 sections is summarized below:

- Transverse cracking has occurred on several JPCP sections, which is probably attributed to the 30-ft (9.1-m) joint spacing.

- The lone section constructed on ATB is showing the best overall performance. It is exhibiting no spalling, little cracking, and the lowest faulting of all sections (even lower than sections that contain dowel bars).

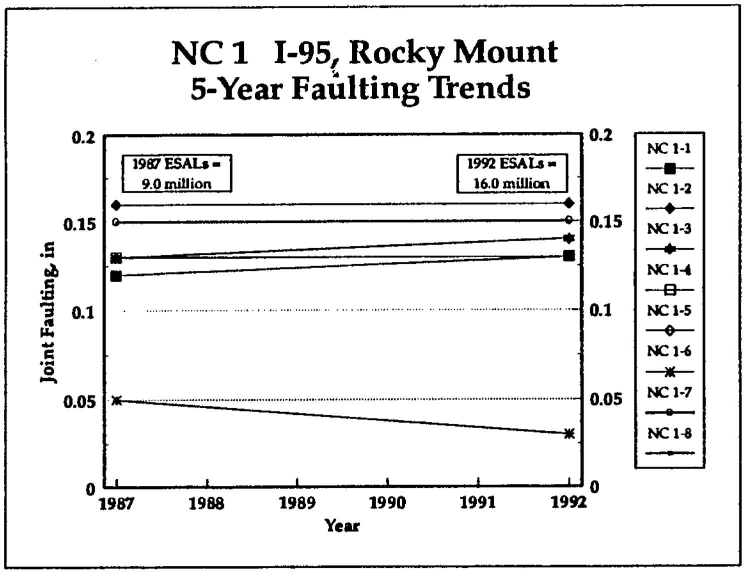
- The sections constructed on cementitious bases were not only susceptible to substantial joint faulting, they also exhibited a significant amount of longitudinal cracking. The sections constructed on soil cement bases in particular exhibited a great deal of longitudinal cracking. This may be due to the high cement content of the base (8 percent) that may have stiffened the base, resulting in increased curling stresses.

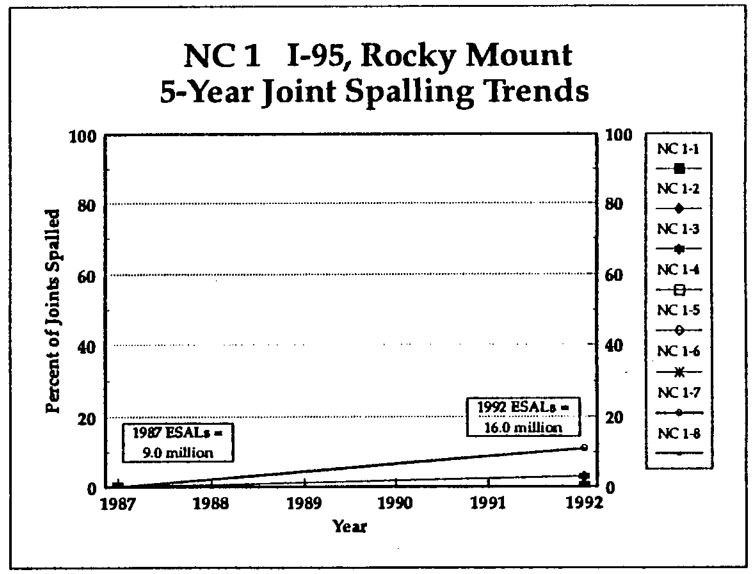
- The pavement sections constructed on aggregate base display significant joint faulting, even those that contain dowel bars. One section displayed a substantial amount of slab cracking that may be due to a subgrade or construction problem.

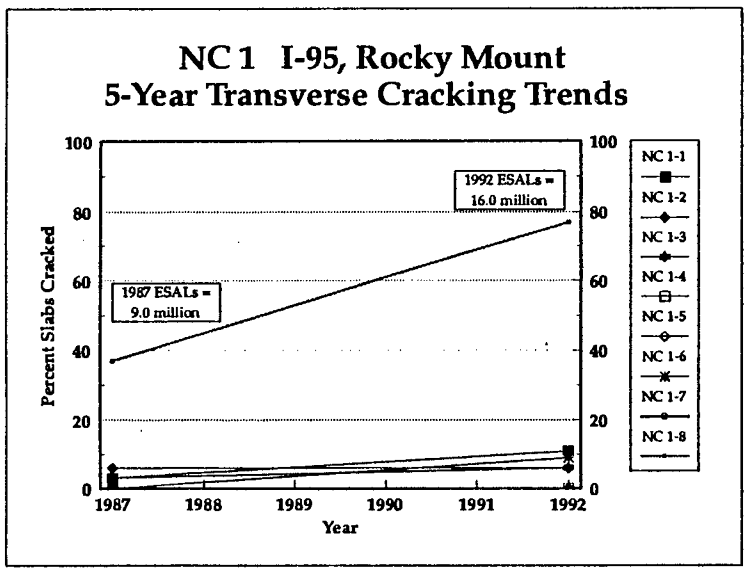
- The 1-in (25-mm) diameter dowel bars were not effective in reducing joint faulting on these sections. The levels of faulting exhibited by the doweled sections is not significantly different from the nondoweled sections. This result is believed to be due to the small diameter of the dowel bars and the 30-ft (9.1-m) joint spacing.

- Skewed joints have been effective at reducing faulting, transverse cracking, and roughness when compare to corresponding sections with perpendicular joints.

- The 8-in (203-mm) JRCP is showing fair to good performance, certainly comparable to several of the 9-in (229-mm) JPCP sections.







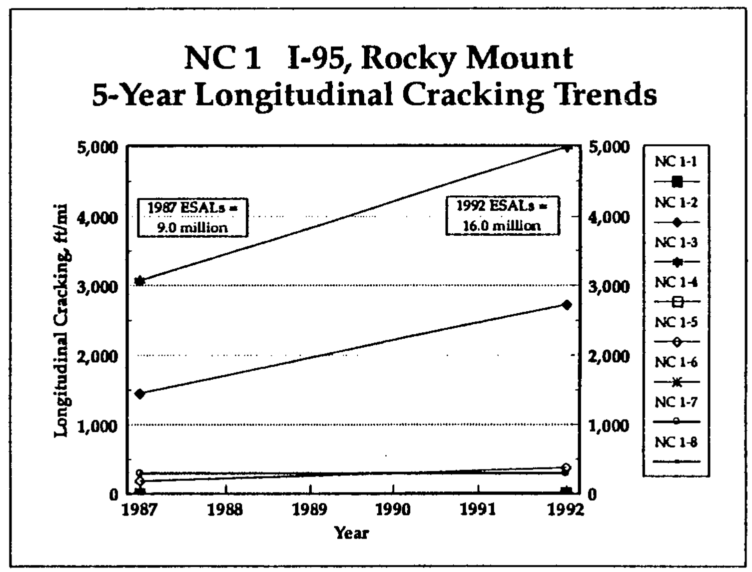


Figure 78. 5-year performance trends for NC 1.

- The JRCP section, with the longer joint spacing, displayed more joint spalling than any of the other sections. However, the amount of joint spalling for all sections is extremely low for a 25-year-old pavement.

- The CRCP section is performing very well. It displayed no structural distresses and elicited a very good serviceability rating. The serviceability rating of the CRCP section was the same as that of the best-performing jointed concrete section (NC 1-6).

- The 5-year performance trends for the NC 1 sections show that all sections have experienced a decrease in serviceability since the 1987 survey. In addition, while joint faulting and joint spalling have not changed significantly since the 1987 survey, one section (NC 1-8) exhibited a marked increase in transverse slab cracking. Furthermore, the two sections constructed on soil cement base show a significant increase in longitudinal cracking since 1987.

North Carolina 2

This single section, located on I-85 near Greensboro, is included in the study because of its doweled, JPCP design and tied concrete shoulders. It is also a section being monitored by the North Carolina Department of Transportation to evaluate the effect of the lean concrete base on pavement performance.[44]

Constructed in 1982, the pavement consists of 11-in (279-mm) JPCP slabs placed on a lean concrete base. The subgrade for the project is an AASHTO A-4 material. The transverse joints are spaced at 18-25-23-19-ft (5.5-7.6-7.0-5.8-m) intervals, contain 1.38-in (35-mm) diameter dowels, and are sealed with a silicone sealant. Fin drains are also provided in the pavement structure.

The design and performance data for the NC 2 section are summarized in table 78. This section is still performing well, with little joint faulting, slab cracking, or joint spalling. Indeed, these distresses have remained virtually unchanged since the 1987 survey. There is some fine, hairline longitudinal cracking near the centerline of the pavement that apparently was not observed in 1987, but this is not adversely affecting the performance of the pavement.

The roughness data and serviceability data indicate a smooth-riding pavement. The serviceability has dropped off some since 1987, but the pavement is still providing good rideability.

Table 78. Design and outer lane performance data for NC 2.

	NC 2	
Pavement Type	JPCP	
Year Built	1982	
Thickness, in	11.0 (279 mm)	
Joint Spacing, ft	18-25-23-19 (5.5-7.6-7.0-5.8 m)	
Dowel Diameter, in	1.38 (35 mm)	
Base Type	5-in (127-mm) LCB (5% cement)	
Subbase	None	
Subgrade	AASHTO A-4	
Performance Data	1987	1992
Age at Survey, years	5	10
ESAL's, millions	5.1	14.2
Faulting, in	0.02	0.02
Det. Tr. Crks/mi	10	10
% Slabs Cracked	4	4
Long. Cracks, ft/mi	0	2608
% Joints Spalled	0	2
PSR	4.2	3.8
IRI, in/mi	—	83

1 in = 25.4 mm
1 ft = 0.305 m
1 mi = 1.61 km

Overall Summary

This chapter presents the performance data for 308 concrete pavement sections surveyed in 1992. The performance of the sections are described and, where possible, comparisons between different design sections are made. The 5-year performance trends of the original 95 sections are also described. Possible explanations for the differences in performance and the performance trends are also offered where appropriate.

The next step in the evaluation of these sections is a nationwide analysis that identifies overall performance trends and, where possible, indicates the effect of design features on PCC pavement performance. This information is presented in volume II.

4. CLOSURE

This report presents a summary of the work that has been accomplished to date under FHWA project DTFH61-91-C-00053, *Performance Evaluation of Rigid Pavements—Data Collection and Analysis.* The goal of this study is to improve the design and performance capabilities of concrete pavements through an examination of the performance of in-service concrete pavement sections.

As described in chapter 2, a total of 270 concrete pavement sections (located in 16 States and provinces) were manually surveyed and evaluated over a 6-month period during the summer of 1992. Most of the sections are part of an experimental project constructed by a highway agency to evaluate the effect of various design features (e.g., dowel bars, base type) on concrete pavement performance. Other sections are included because they incorporate a unique design feature (such as widened traffic lanes). Of the 270 sections, 95 sections were also surveyed and evaluated in 1987, meaning that time series data that can be used to develop deterioration rates and performance trends will be available.

In addition to the data from the 270 pavement sections, data are available from automated survey methods for many of the experimental projects. These data have been added to the project evaluation for sections which were not surveyed using manual methods, yielding a total of 308 pavement sections. The distress data from the automated distress surveys will also be used in the data analysis to evaluate how representative the data from the sample surveys are and to test the preliminary performance prediction models that will soon be developed.

Of the 270 pavement sections evaluated using manual distress survey procedures (to note the occurrence of such items as cracking, spalling, and faulting), many were subjected to a more intensive evaluation using nondestructive and destructive testing methods. A detailed drainage survey was also conducted on each section, along with the measurement of pavement profiles using a South Dakota-type profiler. Uniform and comprehensive data collection procedures were followed to ensure consistency in the data. The field testing procedures, the type of testing that was conducted on each section, the data reduction activities, and the development of the project database, are all described in chapter 2.

The performance data for each of the sections are presented in chapter 3. This information is presented on a project-by-project basis to allow comparisons of the various pavement sections within each project. Where possible, the effect of various design features on concrete pavement performance is identified, and the 5-year performance trends for the original 95 sections are examined.

In the presentation of the data in chapter 3, some attempts were made to explain the performance trends observed within each project. The next step in the data analysis process is to try to extrapolate the findings on a national level and develop performance prediction models.

Several appendixes are included in volume IV, of which the most relevant for this report is appendix A. Appendix A provides a summary of the type of testing that was conducted on each section, as well as the project summary tables, which list the design and performance data for all of the surveyed sections.

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